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Best Management Practices Implementation
Rochester, New York

O'Brien and Gere Engineers, Inc.
Syracuse, NY

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The Best Management Practices(BMPs) offered an attractive and feasible alternative to the partial solution of stormwater runoff induced receiving water quality impairment for the City of Rochester, New York. The configured BMP program resulted in a measureable reduction in the frequency and volume of combined sewer overflow (CSO) discharged to the Genesee River. The study defined and outlined the effective BMP measures, advanced a methodology of approach, and established preliminary cost/benefit relationships.

A program of source control and collection system management BMP concepts proved effective in reducing the frequency and volume of CSO for storm events with rainfall volumes of 0.25 in. or less. For intense storm events the identified system improvements resulted in minimal CSO reductions.

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Organic loading			
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Sewer flushing			
Storm water runoff			

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Best Management Practices Implementation

Rochester, New York

Lake Ontario
Rochester

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concerns about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technologies and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication reflects the application of research, in a project scale demonstration of innovative technology.

In order to support the demonstration of new methods and techniques for the control of pollution within the Great Lakes, Congress provided funds through Section 108 of the Clean Water Act. This project to develop and demonstrate innovative yet practical approaches to solve problems caused by discharges from combined sewers, has been funded by Section 108 through the Great Lakes National Program Office.

The deleterious effects of stormsewer discharges and combined sewer overflows upon the nation's waterways have become of increasing concern in recent times.

This report presents the overall framework for the implementation of Best Management Practices (BMP) concepts for the management of combined sewer overflows from the City of Rochester impacting the Genesee River. The configured BMP program involving source and collection system management options significantly reduces annual pollutant loadings from CSOs and is readily compatible with capital-intensive pollution abatement options that are needed for control of pollutants from large storms.

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Chicago, Illinois

EPA-905/9-81-002
April 1981

BEST MANAGEMENT PRACTICES IMPLEMENTATION
ROCHESTER, NEW YORK

by

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DISCLAIMER

This report has been reviewed by the Great Lakes National Program Office and the Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

ABSTRACT

In light of significant capital and operating costs associated with structurally-intensive storage/treatment pollution abatement alternatives, the application of selected Best Management Practices (BMPs) offered an attractive and feasible alternative to the partial solution of stormwater runoff induced receiving water quality impairment for the City of Rochester, New York. The configured BMP program resulted in a measureable reduction in the frequency and volume of combined sewer overflow (CSO) discharged to the Genesee River. The study defined and outlined the effective BMP measures, advanced a methodology of approach, and established preliminary cost/benefit relationships.

A program of source control and collection system management BMP concepts proved effective in reducing the frequency and volume of CSO for storm events with rainfall volumes of 0.25 in. or less. For intense storm events the identified system improvements resulted in minimal CSO reductions.

Evaluations were conducted on the use of porous pavement, improved street and catchbasin cleaning practices, installation of inlet control devices, and the implementation of minimal structural improvements to the existing sewer collection system. For the City of Rochester, selective use of porous pavement and inlet control devices in conjunction with minimal structural improvements to the main interceptor and overflow regulators, selective trunk sewer rehabilitation, and the installation of control structures at various locations throughout the sewer system proved to be the most effective in reducing the average annual volume of CSO discharged to the Genesee River. The implemented and proposed BMP measures were compatible and complementary with ongoing structurally-intensive abatement programs.

This report was submitted in fulfillment of Federal Great Lake Initiative Grant No. G00533401 by O'Brien & Gere Engineers, Inc. under the partial sponsorship of the U.S. Environmental Protection Agency. This report covered a period from October 1978 to November 1980 and was completed by February 1981.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

AC, ac -- acre
ADT -- average daily traffic
BMP(s) -- Best Management Practice(s)
BOD -- biochemical oxygen demand, 5-day at 20°C
CB -- catchbasin
cfs -- cubic feet per second
CIT -- Cross-Irondequoit Tunnel
CSO(s) -- combined sewer overflow(s)
cy -- cubic yard
DO -- dissolved oxygen

ESTS -- East Side Trunk Sewer
FC -- fecal coliform bacteria
ft -- feet
ft/day -- feet per day
ft/sec -- feet per second
GCO STP -- Gates-Chili-Ogden Sewage Treatment Plant
GVISW -- Genesee Valley Interceptor Southwest
in. -- inches
in.^{2} -- square inches
in.^{3} -- cubic inches
lb -- pound
LF -- linear feet
MA -- milliamp
MA7CD/10, Q7-10 -- minimum average 7 consecutive day flow with 10 year return period
MG -- million gallons
MGD, mgd -- million gallons per day
MGH -- million gallons per hour
ml -- milliliter
mg/l -- milligrams per liter
mmhos/cm -- micro mhos per centimeter
mil $ -- millions of dollars
mi -- miles
mi^2 -- square miles
min -- minute
Pb -- lead
Q -- discharge rate
R -- removal
r -- correlation coefficient

SMSA -- Standard Metropolitan Statistical Area
SOD -- sediment oxygen demand
SPBI -- St. Paul Boulevard Interceptor
SSM -- Simplified Stormwater Model
STP -- sewage treatment plant
SWMM -- USEPA Stormwater Management Model
TIP -- total inorganic phosphorus
TKN -- total Kjedahl nitrogen
TSS -- total suspended solids
V -- volt
WB -- Weather Bureau
wk -- week
WSTS -- West Side Trunk Sewer
yd^2 -- square yards
yr -- year

SYMBOLS

A,a -- cross-sectional areas
h_0, h_1 -- original, final hydraulic head
K -- permeability coefficient
L -- length
t -- time
ln -- natural logarithm

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Mr. John Davis, Deputy Director
Mr. Thomas Quinn, Chief of Technical Operations
Mr. James Stewart, Assistant Engineer
Mr. John Graham, Assistant Engineer

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This report was prepared by O'Brien & Gere Engineers, Inc., Syracuse, New York under the direction of Dr. Cornelius B. Murphy, Jr., Vice President and Mr. Dwight A. MacArthur, Managing Engineer.

SECTION 1

INTRODUCTION

BACKGROUND

The City of Rochester, New York, like many of the older cities throughout the United States and especially those in the Northeast, is served by a combined sewer system. In such a wastewater collection system, sanitary sewage, industrial wastes, and stormwater are all collected and conveyed in the same sewer network. During periods of rainfall and snowmelt the resultant surface runoff enters the sewer system through stormwater inlets and catchbasins. In addition, in most combined sewer systems, the roof drains on homes and buildings are connected to the same system. The stormwater that enters the combined sewer system then combines with the already established sanitary and industrial wastes.

In general, combined sewer systems were designed to adequately convey all sanitary and industrial wastes plus approximately two to three times an equal volume of stormwater. To prevent the adverse effects of excessively surcharged sewers, hydraulic relief was provided by overflow regulating structures installed at various locations within the conveyance system. It has been well documented that during recent years these periodic overflow discharges can seriously degrade the quality of the receiving waters to which they discharge. Over the past ten years a large effort has been expended to characterize the chemical nature of combined sewer overflows (CSO's) and to develop methods and processes to reduce and treat these discharges.

PROBLEM DEFINITION

Within the City of Rochester there are thirteen (13) major CSO points that discharge to the Genesee River and to Irondequoit Bay as shown in Figure 1. Based on previous sewer network modeling utilizing the USEPA developed Storm Water Management Model (SWMM), overflows to these receiving water bodies occur on the average of 66 days annually with an estimated total annual volume of 1900 million gallons (MG) (1). It has been shown that these overflows result in the direct contravention of established water quality standards for the Genesee River, impose heavy nutrient and chemical loadings on the river and Irondequoit Bay, and cause bacterial contamination of the public bathing beaches along Lake Ontario in the vicinity of the mouth of the Genesee River. These CSO's also contribute excessive organic solids to the benthos of the lower reaches of the river (2).

Both receiving water bodies, the Genesee River and Irondequoit Bay, have very little assimilation capacity for wet-weather induced CSO's. A monthly

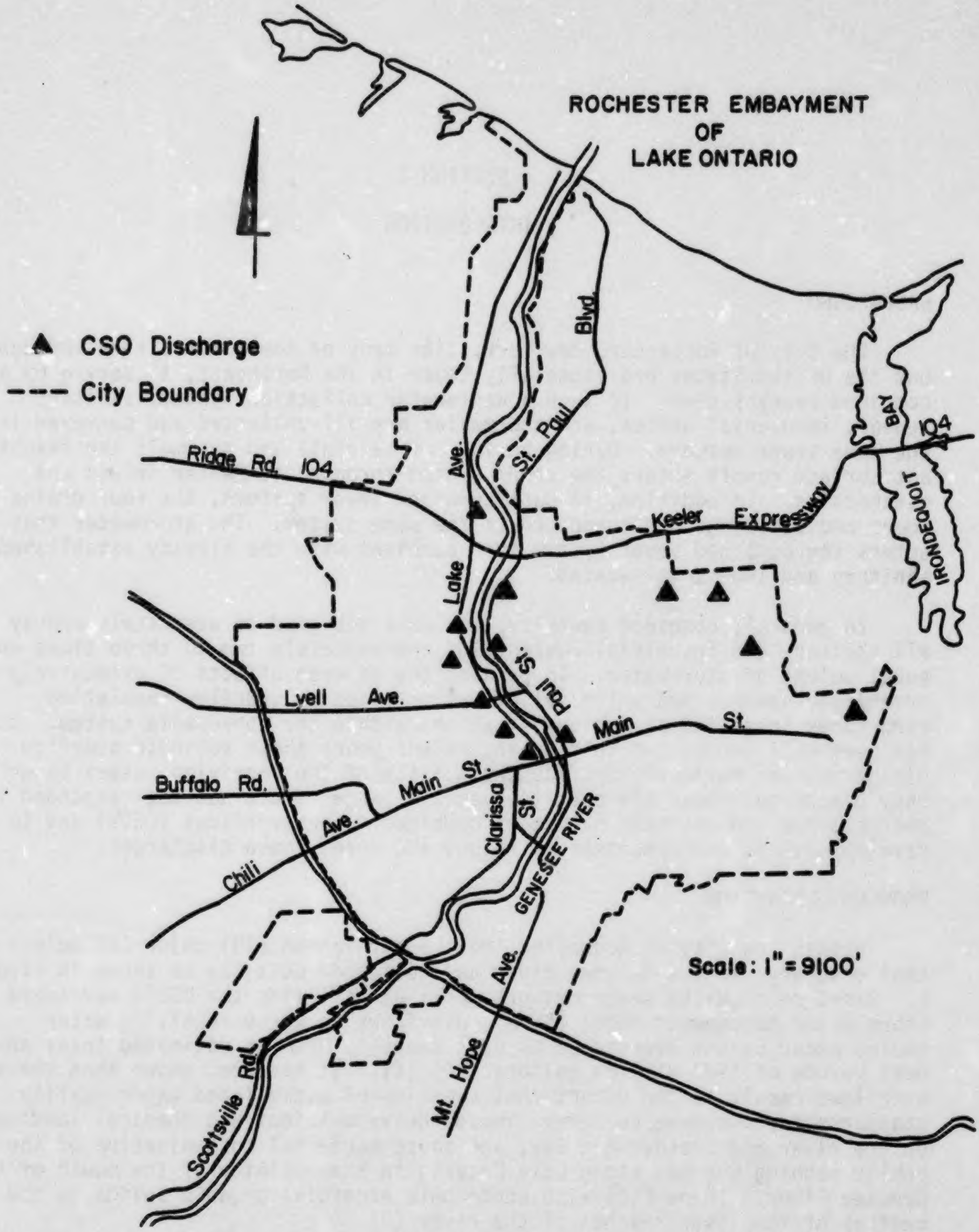

ROCHESTER EMBAYMENT
OF
LAKE ONTARIO
▲ CSO Discharge
---- City Boundary
Ridge Rd. 104
Lake Ave.
St. Paul Blvd.
Keeler Expressway
104
BAY
IRONDEQUOIT
Lyell Ave.
St. Paul St.
Main St.
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
GENESEE RIVER
Mt. Hope Ave.
Scottsville Rd.
Scale: 1" = 9100'

Figure 1. Combined sewer overflow locations within the city of Rochester.

analysis of the assimilation capacity available in the Genesee River indicated probable dissolved oxygen contravention caused by CSO's during the months of June, July, August, and September under MA7CD/10 river flow conditions. In addition, a fecal coliform model derived for the River and the Rochester Embayment of Lake Ontario indicated probable violation of Monroe County Health Department standards for contact recreation at Ontario Beach if a storm event occurs simultaneously with, or is followed by an easterly wind (2). Reduction or elimination of these CSO's would substantially reduce the solids, toxicants, oxygen-demanding constituents, and bacteriological contaminants presently discharging to the River and Bay, leading to a significant overall improvement in water quality.

PROPOSED SOLUTION

In general, CSO abatement technology, to date, has focused on the implementation of structurally-intensive storage/treatment alternatives. The costs involved in many of these capital-intensive programs are enormous. These programs involve the construction of large facilities requiring long-term design and construction periods.

By addressing the sources and causes of pollution, a Best Management Practices (BMP) program can achieve a cost-effective solution to CSO induced receiving water impairment. A BMP approach in conjunction with minor structural improvements to the existing wastewater collection system can substantially decrease the total annual load of contaminants discharged to the receiving waters from wet-weather induced CSO's for relatively small expenditures of manpower and capital. Furthermore, a BMP program can serve as a first phase solution to the CSO problem that can be implemented in a resonably short period of time. Thus, pollution abatement can proceed effectively while long-term design and construction of more structurally-intensive abatement alternatives are undertaken. A BMP program can also provide a long-term partial solution if subsequent funding of capital-intensive programs is interrupted. A BMP program, because of its emphasis on optimized system performance, can possibly result in economy of design of structurally-intensive solutions.

Specifically, a BMP program focuses on the sources of pollutants and their means of conveyance. Integral to a total BMP program is the application of both source and collection system management options. A breakdown of the various elements of a BMP program is shown in Figure 2 (3).

Source management involves the application of measures to reduce or prevent pollution by addressing surface runoff before it enters the conveyance system. Typical source management abatement measures include the application of surface flow attenuation techniques, use of porous pavement, adherence to erosion control practices, restrictions on chemical usage, land use planning, and improved sanitation practices including trash removal and street cleaning.

Collection system management involves the application of abatement alternatives which pertain to the effective management and control of the collection system. Collection system BMP abatement alternatives are those which are applied after runoff enters the collection system. Typical solu-

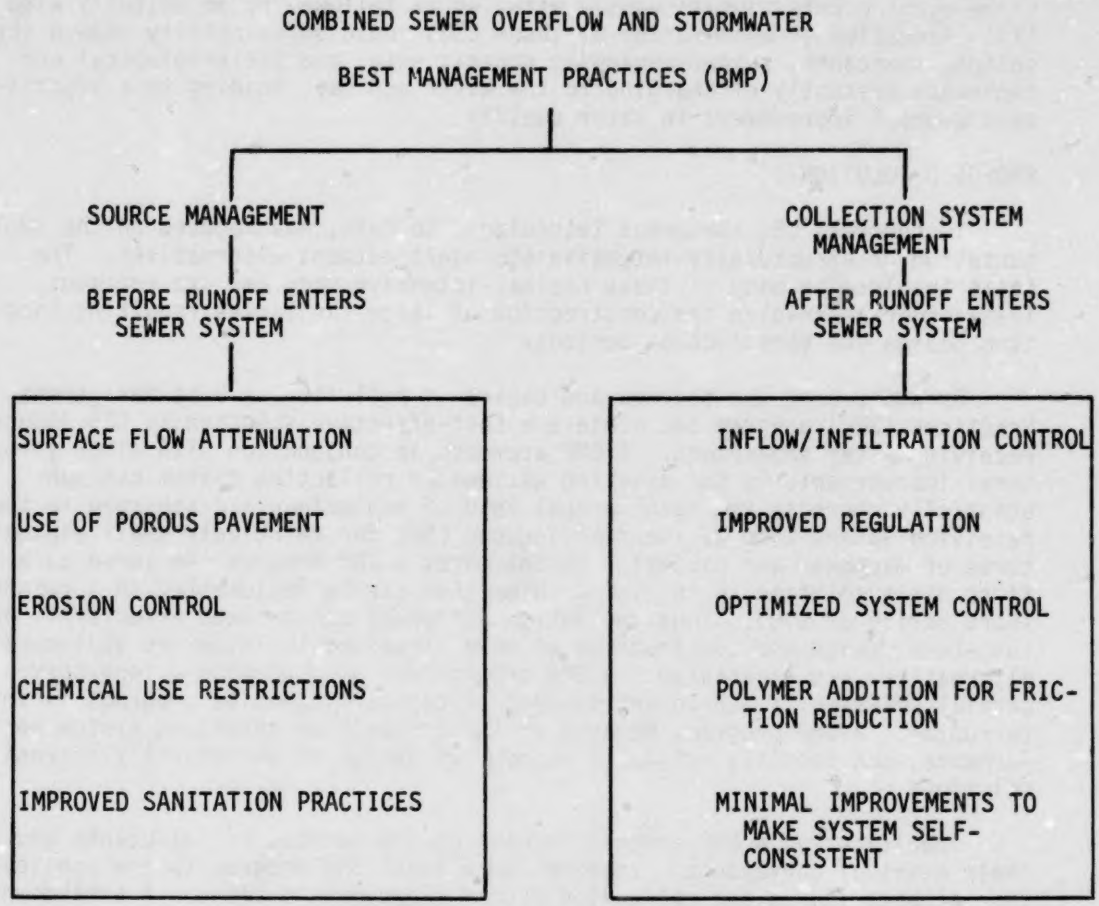

COMBINED SEWER OVERFLOW AND STORMWATER
BEST MANAGEMENT PRACTICES (BMP)
SOURCE MANAGEMENT
BEFORE RUNOFF ENTERS SEWER SYSTEM
SURFACE FLOW ATTENUATION
USE OF POROUS PAVEMENT
EROSION CONTROL
CHEMICAL USE RESTRICTIONS
IMPROVED SANITATION PRACTICES
COLLECTION SYSTEM MANAGEMENT
AFTER RUNOFF ENTERS SEWER SYSTEM
INFLOW/INFILTRATION CONTROL
IMPROVED REGULATION
OPTIMIZED SYSTEM CONTROL
POLYMER ADDITION FOR FRICTION REDUCTION
MINIMAL IMPROVEMENTS TO MAKE SYSTEM SELF-CONSISTENT

Figure 2. Elements of a typical BMP program.

tions fall into two basic categories: structural-intensive and minimal-structural alternatives. The collection system management alternatives, which are of interest in a BMP program, involve those alternatives which, when applied, optimize the effectiveness of other elements of the collection and treatment systems. These alternatives generally require the expenditure of minimal resources. Relevant collection system management practices include inflow/infiltration control, improved system regulation, optimized system control, polymer addition for friction (head loss) reduction, and minimal improvements to make the collection system self-consistent (elimination of conveyance systems throttling constraints).

It is important to note that the effectiveness of implemented BMP measures are most clearly identifiable when measured on an annual basis. The potential reduction in volume and frequency of CSO due to the implementation of a BMP program is greatest when pollutant loadings are examined on an annual, cummulative basis and not on an individual storm event basis. It is also important to recognize that for many parameters that influence receiving water quality, their effect can be best described when evaluated over a long period of time.

Pollutants found in CSO discharges, such as oxygen-demanding substances, toxicants, heavy metals, and suspended solids influence receiving water quality when considered over the long-term. Coliform organisms which are present in all CSOs, however, exhibit relatively rapid die-off rates in natural waters and, therefore, their effects must be calculated over a short period of time, such as during and immediately following a storm event. Average loads for pollutants with short-term transient impacts are, therefore, not as readily addressed by BMP oriented solutions.

The preceeding discussion tends to highlight one very important point: the nature of a particular CSO problem must be thoroughly understood prior to the implementation of any CSO pollution abatement program. The application of BMP control measures may be effective in reducing the annual loads but may be ineffective in reducing short-term transient parameters that contravene established water quality standards. More capital-intensive alternatives may be necessary to satisfy specific water quality needs. In any event, however, a thorough understanding of the existing conveyance and treatment system is required to determine the general applicability of a BMP approach to CSO pollution abatement.

SECTION 2

CONCLUSIONS

Based on the results of this investigation, the following conclusions relative to Best Management Practices for combined sewer overflow pollution control are presented:

General

1. BMP's have been shown to be effective in reducing CSO generated during frequent, low intensity storms; however, more conventional capital-intensive measures may be required to abate CSO pollution for the less frequent, high-intensity storms.

2. BMP collection system concepts are more easily implemented than conventional solutions and tend to maximize the use of the existing conveyance system.

3. Institutional constraints, public acceptability, and the lack of consistent performance make source control concepts less practicable than collection system options as solutions to CSO problems. However, application of selected source control concepts may be used as supplemental measures for stormwater management.

4. During the course of this study, it was found that the following collection system and source control concepts were most effective in reducing the frequency and volume of CSO discharged:
   - Improved system regulation
   - Elimination of conveyance system bottlenecks
   - Split-flow mode of operation at existing treatment facilities under wet-weather conditions
   - Effective utilization of existing in-system storage
   - Porous pavement in parking lot applications, given suitable soil conditions
   - Stormwater inlet control

5. Implementation of BMP concepts in Rochester, NY has resulted in the following water quality and environmental benefits:
   - Reduction of raw wastewater discharges to the area's receiving waters

- An overall improvement in receiving water quality as a result of reduced pollutant loadings from CSOs for many of the storm events that occur in an average rainfall year
- Improvement in the quality of sediment in transit in the receiving waters
- Reduction in volatile solids contributed to the benthos

6. Implementation of BMP measures involves an increased commitment of operating dollars.

7. Substantive hydraulic modeling and thorough field inspection of the collection system and treatment plant capabilities are imperative in determining the applicability of BMP concepts prior to implementation of any collection system management options.

8. Integral to the successful implementation of collection system management concepts is the utilization of a centralized monitoring and performance evaluation function.

9. Adoption of collection system management concepts will generally result in increased hydraulic and pollutant loadings to the existing treatment facilities.

10. Under-utilized conveyance and storage facilities must exist to allow the effective application of a BMP collection system management program. Evaluations must be carried out on a site-specific basis.

Specific to Rochester, New York

Interceptor Improvements

1. Field inspection of the St. Paul Boulevard Interceptor (SPBI) indicates that it is in structurally good condition. There is no evidence of need for rehabilitation other than replacing many of the manhole steps that have deteriorated.

2. Modeling of the SPBI using the USEPA Stormwater Management Model (SWMM) indicated that "bottlenecks" or flow restrictions exist within the overall interceptor system. To fully utilize the potential conveyance capacity of the interceptor, these restrictions must be removed. These restrictive sections specifically include the West Side Trunk Sewer Diversion Sewer siphons originating at the Glenwood Screenhouse, the SPBI siphons from the Cliff Street Screenhouse to Avenue 'B', and the reach of the SPBI immediately downstream of the confluence of those two siphon systems, from Avenue 'B' to Norton Street. Refer to Figure 7 for the location of the SPBI.

3. The design process capacity of the Frank E. VanLare Treatment Facility is 100 MGD. Flows greater than this can result in the washout of biological solids. Flowrates as high as 200 MGD can be adequately processed, however, by allowing the plant to operate

under a split-flow mode of operation, whereby 100 MGD is passed through the settling and biological processes units with the remainder receiving chemically-enhanced settling plus disinfection. The overall quality of the final plant effluent is better than if the split-flow mode of operation was not conducted.

System Regulation

1. Minor adjustments and modifications to the existing overflow regulators and weirs have accounted for CSO volume reductions of 5-30% for intense storms (greater than 0.5 in./hr) and 60-100% for small storm events (0.01 - 0.20 in./hr).

2. Based on stormwater modeling using the USEPA SWMM and the Simplified Stormwater Model (SSM), CSO volume reductions on the order of 41% and duration reductions of 46% can be realized for an average rainfall year upon implementation of selective regulator/weir modifications in conjunction with removal of the identified flow-restrictive segments of the SPBI.

3. As a result of the implemented BMP regulator modifications, a general improvement in the water quality of the Genesee River and the Rochester Embayment of Lake Ontario is expected. Because of a limited data base and short period of time that the identified modifications have been in place, a definitive assessment of water quality improvement cannot be made.

4. Reductions in total pollutant loads of BOD, TSS, TKN, TIP and bacteria are generally consistent with the reduction in overflow volume. This general pattern may be altered due to specific rainfall and antecedent storm conditions.

5. Implementation of BMP identified system improvements and the split-flow mode of operation at the Van Lare treatment plant is projected to result in a reduction in annual TIP loading to the Genesee River and Rochester Embayment of approximately 27 percent.

Porous Pavement

1. Laboratory testing has indicated that a porous asphaltic concrete containing 5.5% asphalt by weight and an open-graded aggregate can exhibit initial permeability rates as high as 1900 in./hr.

2. A field demonstration showed that the peak rate of runoff from a parking lot utilizing porous pavement with a underdrain system was reduced by approximately 83% in comparison with the peak runoff rate from conventional pavement utilizing storm inlets and sewers.

3. A core sample of porous pavement was subjected to 100 freeze/thaw cycles with no observable structural degradation.

4. Laboratory testing of permeability rates of porous pavement indicated that repeated loadings of sand are required to induce a significant reduction in permeability. Even after loadings simulating 6 yr of adding roadway abrasive materials, the permeability rate exceeded the highest rainfall intensities likely to be observed.

5. The Lake Avenue porous pavement demonstration site showed that significant clogging (reduction of 94% from initial permeability rates) of porous pavement can occur if overland runoff carrying sediment is allowed to pass onto the pavement. The structural integrity of the porous pavement at this site remained unimpaired under heavy and frequent traffic loadings.

6. The cost of constructing a porous pavement parking lot utilizing an impermeable membrane and underdrains $18/yd^2 is slightly higher than that of a conventionally paved lot with stormwater inlets and subsurface piping $16/yd^2. If subsurface soil conditions are adequate to allow passage of the rainfall that infiltrates through the porous pavement, then the impermeable membrane and underdrains are not needed and costs for both types of pavements would be the same.

Trunk Sewer Evaluations

1. Field inspection of the West Side Trunk Sewer (WSTS) from Alice and Glide Streets to Glenwood Avenue and Malvern Street indicated that the original tunnel is in good condition. No significant structural deficiencies or excessive sediment accumulations were observed.

2. The East Side Trunk Sewer (ESTS) inspection indicated that significant grit has accumulated in the brick pipe section of the ESTS from Edgeland and Rocket Streets to Waring Road and Norton Street. The grit depth varied from 12 to 24 in. Within this section of the ESTS the sewer size varied from 5.5 to 6.0 ft in diameter.

3. Major structural deficiencies were observed in the unfinished rock-tunnel portion of the ESTS along Norton Street. Excessive sediment and debris were noted in the ESTS immediately east of both Portland Avenue and Clinton Street. Significant spaulling was also observed at numerous locations in the ESTS along Norton Street. These factors contributed to a loss in potential in-system storage volume of about 20% and in conveyance capacity of approximately 65%.

4. The total estimated in-system static storage volume is 3.5 MG, which can be effectively utilized only after the installation of control structures/regulators at various locations within the existing conveyance system.

SECTION 3

RECOMMENDATIONS

Based on the results of this investigation, the following recommendations relative to Best Management Practices for combined sewer overflow pollution control are presented:

General

1. Thorough investigations of BMP options should be conducted prior to adoption and implementation of capital-intensive structural alternatives.
2. Water quality data should be collected to define urban runoff problems and the effectiveness of various BMP abatement measures.
3. Intensive efforts should be expended in post-implementation evaluations of adopted BMP abatement measures to determine their effectiveness. Communities should recognize that a reduction in capital outlays for a BMP oriented program will require a long-term commitment of dollars for system operation and maintenance in order to maximize BMP effectiveness.
4. Priority should be given to Construction Grants review and approval of BMP programs.
5. USEPA and state agencies should give strong consideration to funding innovative source and collection system management options for CSO control.
6. The method of operating treatment facilities under a split-flow mode should be adopted for facilities having little or no wet-weather capacity. The USEPA should review treatment plant effluent percent removal requirements.
7. A handbook for implementation of BMP measures should be prepared and distributed to allow practitioners to assess the applicability of such measures for their specific needs.
8. Consideration should be given to the utilization of porous pavement in redeveloping residential, commercial and industrial parking areas. A thorough evaluation of existing soil conditions is required.

9. Stormwater inlet control practices should be closely evaluated as a means of reducing the hydraulic peaks in a combined sewer system, which can result in basement backups as well as CSO's.

10. Every effort should be made to thoroughly understand the existing collection, conveyance, and treatment systems prior to implementing those BMP measures that are intended to maximize the use of the existing systems.

11. A detailed analysis of the impacts of conveying additional hydraulic and pollutant loads to the treatment facilities as a result of implementing BMP measures should be conducted. For example, sludge handling and disposal needs and process limitations should be investigated to determine the overall applicability of contemplated system improvements.

12. A water quality monitoring program should be established to provide continuous feedback of water quality improvements to adequately assess both short and long-term benefits of implemented BMP programs.

13. New permits for sewage treatment plants (STPs) treating wet and dry-weather flows should consider allowances for wet-weather pollutant discharge concentrations apart from overall percent removal requirements established for dry-weather conditions. For example, elimination of the 85% removal requirement for secondary treatment plants would allow the permittee to operate the treatment facility in such a way as to optimize the overall plant pollutant removal capabilities under increased hydraulic loadings encountered during storm events.

Specific to Rochester, New York

1. The overflow and in-system flow monitoring data acquisition system should be maintained and operated for the purpose of:

   - Determining the frequency and magnitude of CSO's discharging to the Genesee River.

   - Identifying those sections of the trunk and intercepting sewer system that are not fully utilized during storm events to allow further readjustment of regulators and weirs to optimize system operation.

   The present rain gauge system does not need to be further maintained and operated. A more sensitive rain gauge monitoring system will be required for overall CSO system control to be implemented under other ongoing programs.

2. The identified flow-restrictive segments of the SPBI should be upgraded so that an optimized, self-consistent flow regime can be maintained in order to reduce overflow to the Genesee River. A

USEPA Construction Grants Step 1 study was received by the Rochester Pure Waters District. Step 2 funding should be sought.

3. The Frank E. VanLare Treatment Facility should be operated under a split-flow mode of operation to maximize treatment of wet-weather flows, thereby minimizing the impact of plant effluent on the Rochester Embayment of Lake Ontario. The plant's permit should be altered to eliminate the 85% removal requirement to allow the needed flexibility in plant operation which would result in enhanced plant effluent quality. The 30-30-1 (BOD, TSS, TIP) requirement should be maintained.

4. The East Side Trunk Sewer (ESTS) should be substantially rehabilitated between Jewel Street and Waring Road. That portion of the ESTS from Waring Road to Garson Avenue should be cleaned to remove excessive grit deposition. After rehabilitation, CSO's presently discharged to Irondequoit Bay from the ESTS will be substantially reduced. A USEPA Construction Grants Step 1 study was received by the Rochester Pure Waters District. Step 2 funding should be sought. Periodic tunnel inspections should be part of the overall improvement program.

5. Further analysis should be conducted on the potential use of ESTS in-system storage. Work should include consideration of by-pass volumes at Atlantic and Garson Avenues and at Densmore Creek. After rehabilitation, evaluations should again be conducted to determine the additional in-system storage volume realized.

6. A continuous review of wastewater levels in the major trunk sewers should be made to insure that, as a result of the modified overflow regulators and weirs, adverse backwater and surcharge conditions do not occur during periods of rainfall.

7. Review of the effectiveness of the implemented regulator/weir modifications should be continued. Consideration should be given to replacing the hardware that was installed on an interim basis with permanent controllable gates. A USEPA Construction Grants Step 1 study was received by the Rochester Pure Waters District. Step 2 funding should be sought.

8. Consideration should be given to keeping the identified high-polluting industrial discharge associated with the Carthage drainage area within the trunk and intercepting sewers, and not allowing it to become part of the CSO from this service area. Although structural improvements will be necessary to accomplish this, in the interim prior to construction of required facilities, the Carthage regulator should be modified by removing the orifice plate/float mechanism. This would result in an immediate increase in the wastewater transfer rate to the SPBI of 8 cfs (5 mgd) or a 47% increase over the present transfer rate.

9. The present street sweeping operations conducted by the City of Rochester Department of Environmental Sanitation should be continued. Increased street sweeping frequency is not a viable BMP method to reduce the pollutional loads on the receiving waters by CSO's. The current level of street cleaning effort is adequate.

10. The current levels of sewer cleaning and periodic sewer maintenance provided by the Rochester Pure Waters District should be continued.

11. Consideration should be given to the use of porous pavement in all redevelopment projects involving parking areas. A site-specific soil suitability review should be conducted as part of each application.

12. Further field testing of Hydro-Brake static flow regulators should be conducted prior to adoption of a area-wide implementation program.

13. Consideration should be given to the use of inlet control devices as a means of reducing the peak stormwater inflow rate into the sewer collection system. This should be done after a site-specific review of the potential for storing additional stormwater runoff in the streets and gutters.

14. Dynamic control devices with associated instrumentation should be installed on the combined sewer system at the following locations:

    - Norton Street and Portland Avenue
    - Norton Street and Waring Road
    - East Side Trunk Sewer at Jewel Street
    - Lexington Avenue Tunnel at the overflow regulator
    - West Side Trunk Sewer at the overflow regulator
    - Front Street Sewer at the diversion point to the interceptor
    - Genesee Valley Interceptor Southwest at the connection to the Clarissa Street Tunnel

    A USEPA Construction Grants Step 1 study was received by the Rochester Pure Waters District. Step 2 funding should be sought.

16. A continuous Genesee River water quality monitoring program should be established to supplement ongoing river sampling programs by the Monroe County Health Department, to further identify the long-term effectiveness of the implemented BMP measures. Such programs should be fundable under the USEPA Construction Grants program.

SECTION 4

STUDY AREA BACKGROUND

GENERAL

The City of Rochester, county seat of Monroe County, is located in the western portion of New York State and borders on the south shore of Lake Ontario as shown in Figure 3. The major receiving water bodies in the area, in addition to the lake, are the Genesee River, which roughly bisects the city, and Irondequoit Bay, which lies to the northeast. The city occupies 35.5 mi^2 of the county's total 763 mi^2. Topography within the urban portion of the city is generally flat and slopes toward the Genesee River from the west and east. In the southern portions of the city and county, gently rolling hills are common. The Genesee River essentially divides the urban center into two distinct drainage areas. The western component drains towards the river from the west and the eastern component drains towards the bay.

Based on preliminary 1980 U.S. Census data, the population of the city is 242,000 with a SMSA population of 939,000 (4,5). The nearly 1,000,000 people within the SMSA places it number 38 in the country. Land use varies widely throughout the city but most industrial concerns are located in the western and northwestern portions. Table 1 presents a breakdown of the general land use categories for the City of Rochester (2).

TABLE 1. SUMMARY OF LAND USE FOR CITY OF ROCHESTER

LAND USE CATEGORY	AREA (ac)	% OF TOTAL
Residential	8200	35
Commercial	1200	5
Industrial	2800	12
Public/Semi-public	4700	20
Streets/Highways	4000	17
Open areas	2500	11

PREVIOUS STUDIES

Periodic CSO and stormwater discharges from the City of Rochester have resulted in the direct contravention of established water quality standards for the area's receiving water bodies. They have caused environmental and

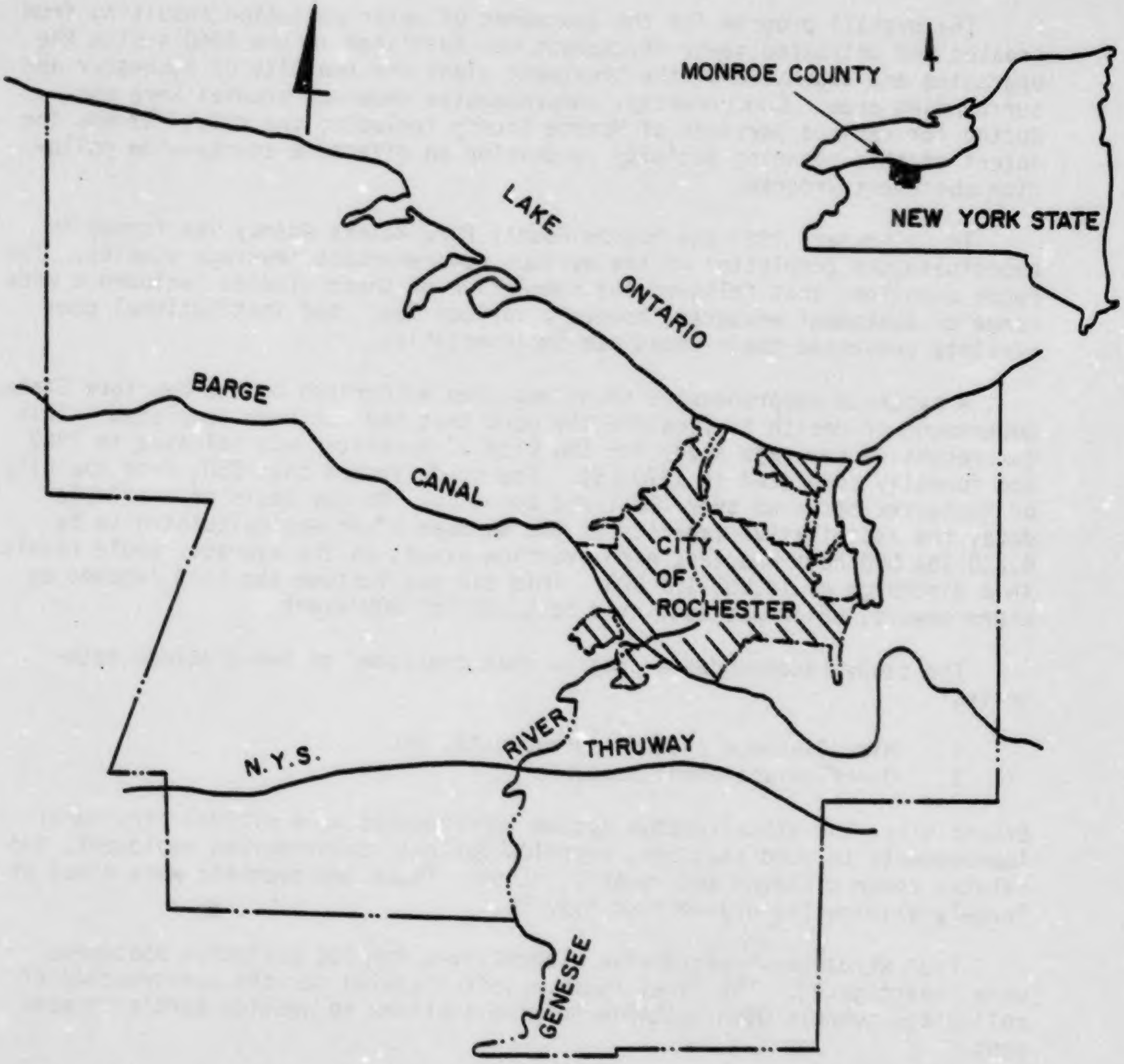

MONROE COUNTY
NEW YORK STATE
LAKE
ONTARIO
BARGE
CANAL
CITY
OF
ROCHESTER
N.Y.S.
THRUWAY
RIVER
GENESEE

Figure 3. Study area location map.

socio-economic problems for which substantial efforts by various governmental agencies have been made to identify the extent of the problems and to implement measures to mitigate their impacts. Goals have been established by Monroe County, the New York State Department of Environmental Conservation, the U.S. Environmental Protection Agency, the International Joint Commission, and local environmental groups.

The overall program for the abatement of water pollution resulting from treated and untreated sewer discharges was initiated in the 1960's with the upgrading and expansion of the treatment plant for the City of Rochester and surrounding area. Concurrently, comprehensive sewerage studies were conducted for various portions of Monroe County including the city. It was the intent of this planning activity to develop an effective county-wide pollution abatement program.

In September, 1967 the Monroe County Pure Waters Agency was formed to coordinate the completion of the various comprehensive sewerage studies. The recommendations that followed the completion of these studies included a wide range of abatement measures; however, various legal and institutional constraints prevented their immediate implementation.

A separate comprehensive study was then authorized by the New York State Department of Health to finalize the work that had not been completed. This comprehensive sewerage study for the City of Rochester was released in 1969 and formally completed in 1970 (6). The study showed that CSO's from the City of Rochester occurred over 100 times annually. On the basis of available data, the assimilative capacity of the Genesee River was calculated to be 8,230 lbs BOD/day; whereas, each overflow event, on the average, would result in a discharge of 14,000 lbs BOD. This did not include the load imposed by storm sewers, which was estimated at 1,330 lbs BOD/event.

The study recommended a program that consisted of two distinct categories:

1. Miscellaneous system improvements, and
2. Overflow abatement measures

Essentially, the miscellaneous system improvements were minimal-structural improvements to pump stations, overflow devices, chlorination equipment, and related sewer cleaning and rehabilitation. These improvements were aimed at largely eliminating dry-weather overflows.

Four structurally-intensive alternatives for CSO pollution abatement were investigated. The final recommendation called for the construction of collection tunnels with multiple holding stations to provide partial treatment.

The City of Rochester then authorized the preparation of an engineering report to review the recommended projects. The report was completed and submitted in 1971 and recommended the construction of holding basins.

Concurrently with the authorization of the City Comprehensive Sewerage Study, the Monroe County Pure Waters Agency authorized a study to review and evaluate various comprehensive studies relating to different portions of the County and to develop an implementable master plan for the county. This study was completed in 1969 and was the basis for the formation of the various County Pure Water Districts (7). One of the recommended projects, the Cross-Irondequoit Tunnel, designed to serve the Irondequoit Bay Pure Waters District, was of major importance to the County's Pure Waters Program and to the City of Rochester. The tunnel has subsequently allowed the phasing-out of 16 sewage treatment plants located throughout the County, the abandonment of a major pumping station, and a reduction in CSO discharges to Irondequoit Bay.

A second important administrative event occurred in 1971 when Monroe County formed a separate Rochester Pure Waters District. The District's major emphasis was placed on the review and implementation of the Combined Sewer Overflow Abatement Program originally initiated by the City of Rochester. The District authorized an engineering study to investigate the feasibility of integrating portions of the original multiple holding basin plan into an east side of the city CSO abatement plan.

The analysis compared the economics of the multiple holding tank plan with those of deep tunnel storage and conveyance for achieving the desired level of CSO pollution abatement. This study demonstrated that a storage-conveyance tunnel system was a more economical method than east side holding basins for eliminating the CSO problem. The initial size of the Irondequoit Tunnel was increased and the east side holding basins were eliminated.

Two detailed drainage basin studies were also completed in the early 1970s. They were the East Side Trunk Sewer Study and the Engineering Report for the Genesee Valley Interceptor (8,9). From these studies, individual projects have been identified and have advanced to final design and construction.

In the mid 1970's a Wastewater Facility Plan (WFP) was completed that integrated all previous and ongoing overflow pollution abatement projects (2). At approximately the same time, a Combined Sewer Overflow Abatement and Pilot Plant Study was initiated and completed. This study was funded as a Research and Development grant from the USEPA and dealt with establishing the magnitude of Monroe County's CSO problem including treatment processes and other measures to alleviate the problem (1). Data and results from this study were used in formulating the Wastewater Facility Plan.

The WFP reported that:

1. The water quality of the area's receiving waters was being adversely affected by CSOs from the Rochester Pure Waters District. The overflows contributed to:
    a. The eutrophication of Irondequoit Bay,
    b. The depression of dissolved oxygen levels in the lower Genesee River, and

   c. The bacterial contamination of Lake Ontario beaches.

2. The sewer network was inadequate to convey the stormwater flows being generated from the present urban environment. The resulting localized surface and basement flooding, caused by backflow of combined sewage, was a public health hazard.

3. Major structural improvements to the sewerage system were needed to:

   a. Consistently meet the water quality objectives of the study area's receiving waters, and
   b. Improve the conveyance capacity of the sewer network to reduce flooding and the associated public health hazards.

4. Minimal and nonstructural alternatives would not solve the identified problems but may serve to enhance the beneficial effects of structural alternatives.

5. The most cost-effective structural improvement for CSO pollution abatement was a tunnel storage/conveyance system.

6. The storage/conveyance system was also the primary element of the plan for upgrading the conveyance capacity of the sewer network to reduce flooding.

In view of the short time frame required for implementation and the modest capital costs associated with a number of minimal and nonstructural alternatives, the Rochester Pure Waters District applied for and received a Section 108 grant from the USEPA Great Lakes Program, Region V to further evaluate and demonstrate selected BMP measures. The resulting BMP program demonstrated the general cost-effectiveness of implementing BMPs to reduce the frequency and volume of CSO on an annual basis. It was intended as an interim program to abate pollution while long-term design and construction of recommended structurally-intensive programs were completed. The management options identified herein as most effective in reducing CSO discharges were supplemental and completely compatible with the ongoing structurally-intensive abatement programs.

DRAINAGE AREA DESCRIPTION

The combined sewer system of the City of Rochester is the focus of the BMP program. All of the present CSO discharges to the Genesee River and Irondequoit Bay originate within city limits. Numerous stormwater outlets also discharge to these receiving water bodies. The various drainage basins and the areas tributary to them are shown in Figure 4. Table 2 summarizes the location and size of the drainage areas within the city tributary to each major receiving water body.

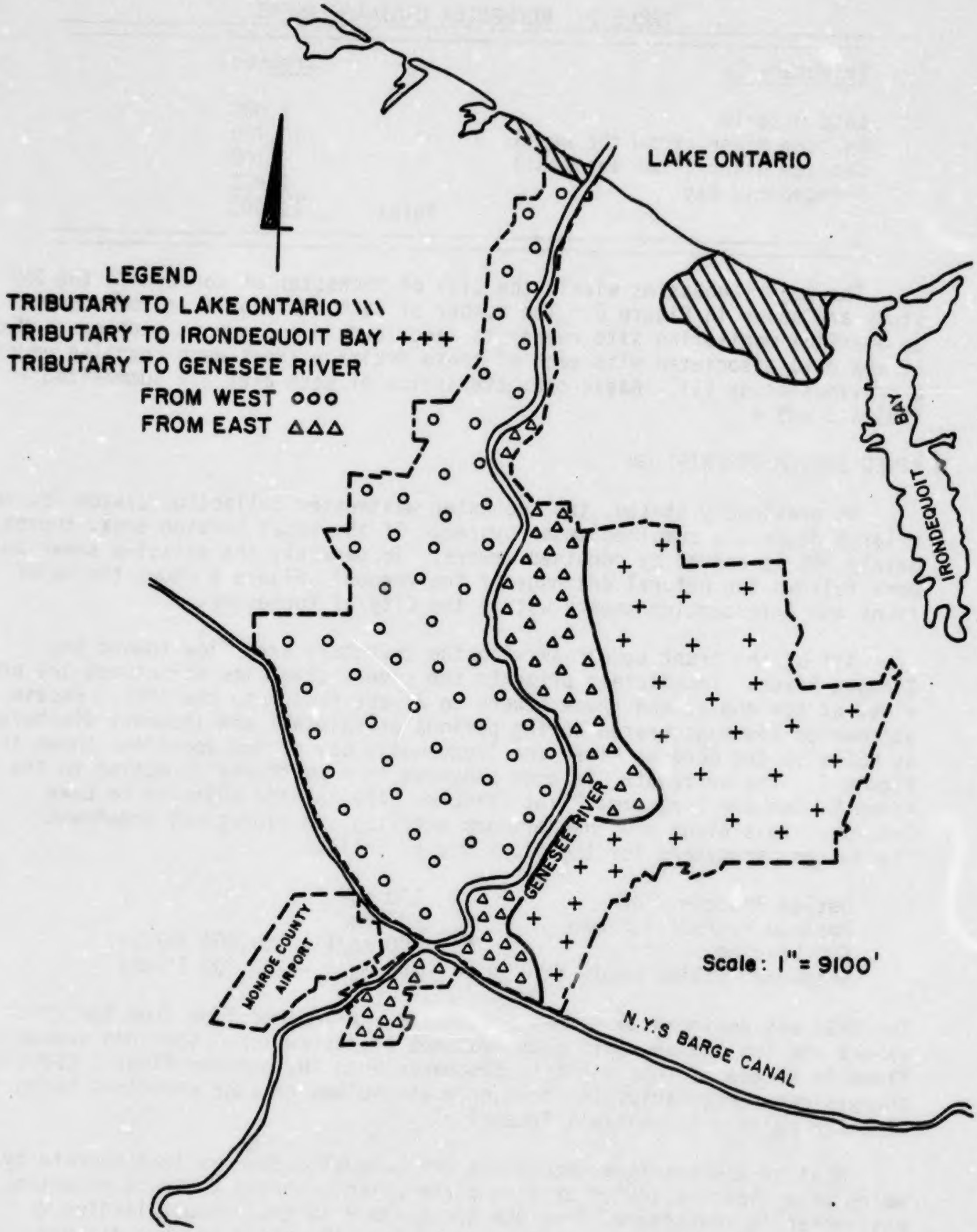

LAKE ONTARIO
LEGEND
TRIBUTARY TO LAKE ONTARIO \\\
TRIBUTARY TO IRONDEQUOIT BAY +++
TRIBUTARY TO GENESEE RIVER
FROM WEST ○○○
FROM EAST △△△
IRONDEQUOIT BAY
GENESEE RIVER
MONROE COUNTY AIRPORT
Scale: 1"= 9100'
N.Y.S BARGE CANAL

Figure 4. Drainage areas within the City of Rochester.

TABLE 2. ROCHESTER DRAINAGE AREAS

Tributary To		Area(ac)
Lake Ontario		1,000
Genesee River (from the west)		10,700
Genesee River (from the east)		3,100
Irondequoit Bay		7,800
	Total	22,600

The major subbasins within the City of Rochester of concern to the BMP study are shown in Figure 5. The number of each drainage area corresponds to overflow monitoring site number as described in subsequent sections. Much of the data associated with each of these drainage areas were compiled under a previous study (1). Basic characteristics of each area are summarized in Tables 3 and 4.

SEWER SYSTEM DESCRIPTION

As previously stated, the Rochester wastewater collection system is, to a large degree, a combined sewer system. Of the total service area, approximately 75% is served by combined sewers. In general, the existing sewer network follows the natural drainage of the region. Figure 6 shows the major trunk and intercepting sewers within the City of Rochester.

All of the trunk sewers serving the tributary area flow toward the Genesee River. Immediately prior to the river, diversion structures are provided at the end of the trunk sewers to divert flow into the SPBI. Excess stormwater flows generated during periods of rainfall and snowmelt discharge as CSO's to the Genesee River and Irondequoit Bay at the locations shown in Figure 1. The wastewater is then conveyed in a northerly direction to the Frank E. VanLare Treatment Plant (VanLare STP) located adjacent to Lake Ontario. This plant provides primary settling and biological treatment. The design parameters for the plant are as follows:

Design Process Flow	100 mgd
Maximum Hydraulic Flow	200 mgd
BOD Loading	300 mg/l - 250,000 lb/day
Suspended Solids Loading	300 mg/l - 250,000 lb/day

The SPBI was designed to convey all of the dry-weather flow from the trunk sewers and two and one-half equal volumes of stormwater. Combined sewage flows in excess of this quantity discharge into the Genesee River. CSO discharges occur regularly, the frequency and volume of such overflows being directly related to rainfall intensity.

Most of the overflow regulators are automatic devices that operate by means of a float-activated orifice plate which controls the rate at which wastewater is transferred from the trunk sewer to the conduit leading to the SPBI. Adjustment of the float mechanism allows for various discharge rates. Associated with each overflow structure is also a weir located within

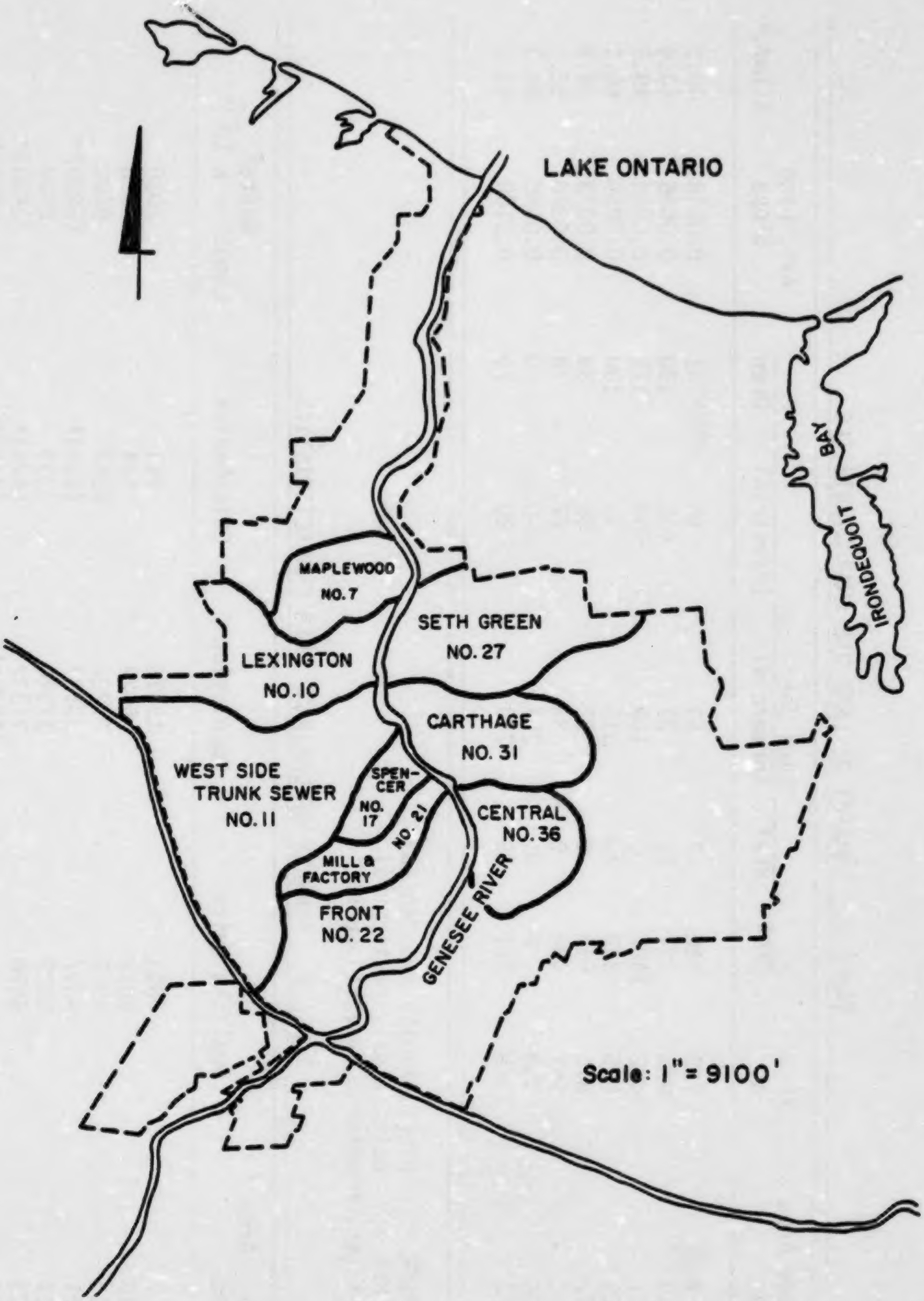

LAKE ONTARIO
IRONDEQUOIT BAY
MAPLEWOOD NO. 7
LEXINGTON NO. 10
SETH GREEN NO. 27
CARTHAGE NO. 31
WEST SIDE TRUNK SEWER NO. 11
SPENCER NO. 17
NO. 21
MILL & FACTORY
CENTRAL NO. 36
FRONT NO. 22
GENESEE RIVER
Scale: 1" = 9100'

Figure 5. Major subbasins tributary to Genesee River within City of Rochester.

TABLE 3. SUMMARY OF LAND USE CHARACTERISTICS

Drainage Area No.	Area ac	Land Use - ac					Ave. Land Slope	% Imp.#
		SFR*	MFR+	Commercial	Industrial	Open		
7	729	610	7	53	18	41	0.0118	48.2
10	988	339	22	32	465	130	0.0066	46.4
11	2682	1407	0	109	994	172	0.0060	49.2
21	826	348	10	318	50	100	0.0059	66.1
27	810	644	0	72	55	39	0.0073	44.9
31	569	434	0	38	27	70	0.0070	52.9
36	348	0	91	257	0	0	0.0080	80.3
22	1169	821	52	200	20	76	0.0100	42.0

*SFR = single family residential
+MFR = multi-family residential
#% Imp. = percent imperviousness

TABLE 4. SUMMARY OF DRAINAGE AREA CHARACTERISTICS

Drainage Area No.	Dwelling Units	Population	Catchbasins	Gutter Length - x 10^{-2}
7	4653	18476	911	2020
10	2016	5522	574	1350
11	9919	27296	2357	6480
21	4397	13687	(826)*	(2800)*
27	6333	17540	1339	2900
31	4596	14332	(626)*	(1450)*
36	0	(12700)*	(435)*	(650)*
22	9444	35088	(1400)*	(3600)*

*Values in ( ) indicate extrapolated or assumed data based on the other drainage areas.

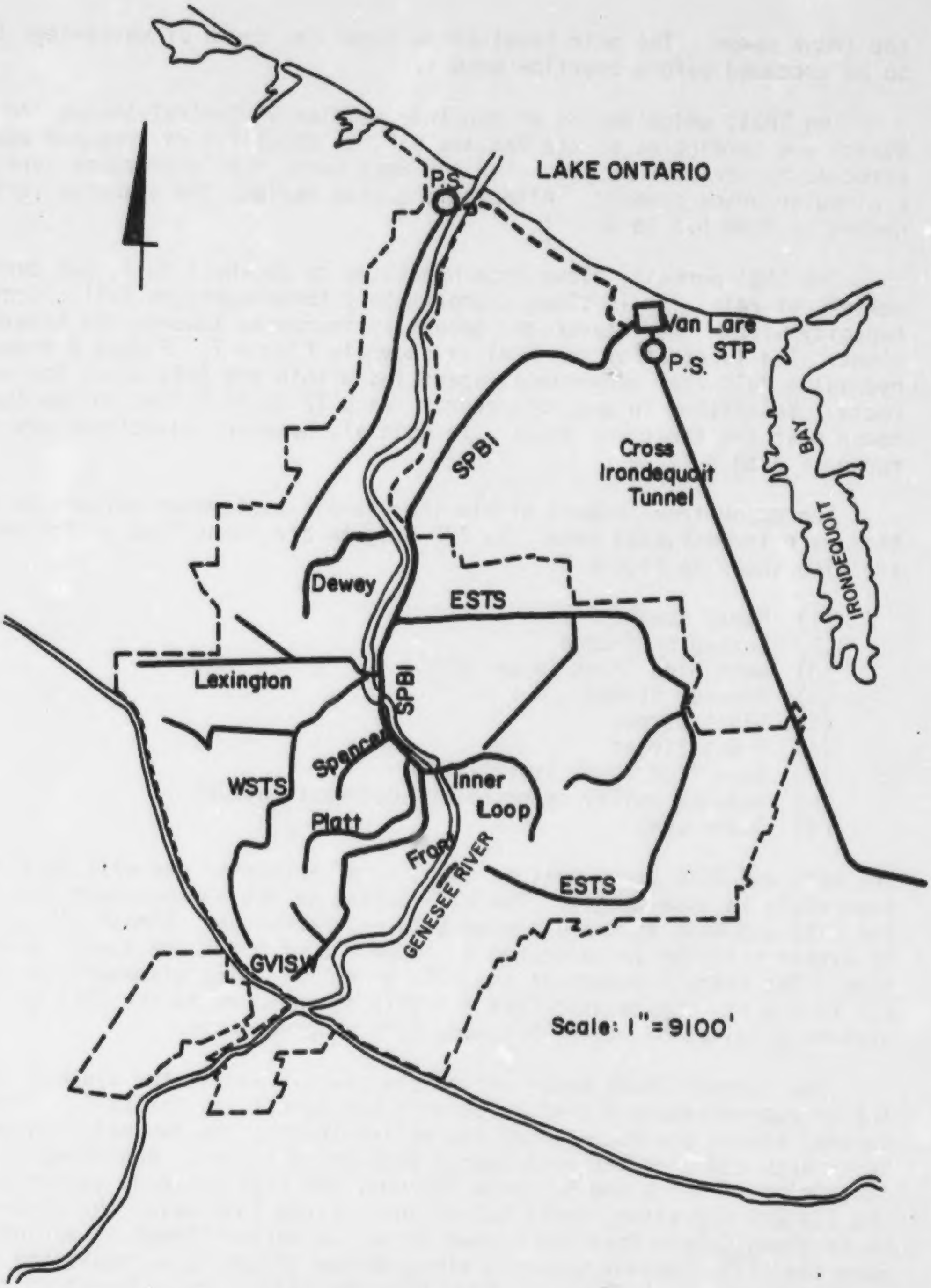

LAKE ONTARIO
P.S.
Van Lare
STP
P.S.
Cross
Irondequoit
Tunnel
BAY
IRONDEQUOIT
SPBI
Dewey
ESTS
Lexington
SPBI
Spencer
WSTS
Inner
Loop
Platt
Front
GENESEE RIVER
ESTS
GVISW
Scale: 1"=9100'

Figure 6. Network of major trunk sewers and interceptor for City of Rochester.

the trunk sewer. The weir level establishes the depth of wastewater that has to be exceeded before overflow occurs.

The SPBI, which begins at the intersection of Central Avenue and Water Street and terminates at the VanLare STP, is about 7.9 mi long and was constructed by various methods. For the most part, the interceptor consists of a circular brick conduit. Although the size varies, the diameter varies generally from 5.5 to 8.0 ft.

The SPBI normally flows from one-third to one-half full, but during periods of rainfall, it flows approximately three-quarters full. Conveyance capacity within the interceptor generally increases towards the treatment plant. The location of the SPBI is shown in Figure 7. Figure 8 shows the hydraulic full-flow conveyance capacities within the SPBI along its entire route. Velocities in the interceptor reach 12 to 14 ft/sec in the lower reach near the treatment plant. In general, however, velocities are in the range of 3 to 5 ft/sec.

The major trunk sewers within the overall wastewater collection system that were investigated under the BMP program are identified as follows and are also shown in Figure 6:

(1) Dewey Avenue
(2) Lexington Avenue
(3) West Side Trunk Sewer (WSTS)
(4) Spencer Street
(5) Platt Street
(6) Front Street
(7) East Side Trunk Sewer (ESTS)
(8) Genesee Valley Interceptor Southwest (GVISW)
(9) Inner Loop

The WSTS and ESTS serve most of the City of Rochester and will be discussed separately in some detail. The WSTS begins in the southwestern portion of the city and ends at the Glenwood Avenue Screenhouse. Almost all of the WSTS is either circular or horseshoe in shape and of brick and tunnel construction. The overall length of the WSTS is 4.3 mi. with diameters varying from 2.0 to 8.0 ft. Wastewater from the WSTS is conveyed to the SPBI by three siphons originating at the Glenwood Avenue Screenhouse.

The longest trunk sewer within the overall wastewater system, consisting of approximately 8.2 mi of conduit and tunnel, is the ESTS. Although several shapes are found along its entire length, the two major shapes are brick arch and circular rock tunnel with brick invert. Beginning at the intersection of South and Mt. Hope Avenues, the ESTS proceeds easterly along the Eastern Expressway until Culver Road. From this point the sewer runs north along Culver Road and Lyceum Street to Norton Street at Waring Road. Here the ESTS proceeds westerly along Norton Street to a regulating structure at Norton and Jewel Streets. Flow from the ESTS is transferred to the SPBI by the regulator chamber. Of importance are four major overflow points on the ESTS which presently discharge to Irondequoit Bay as shown in Figure 1.

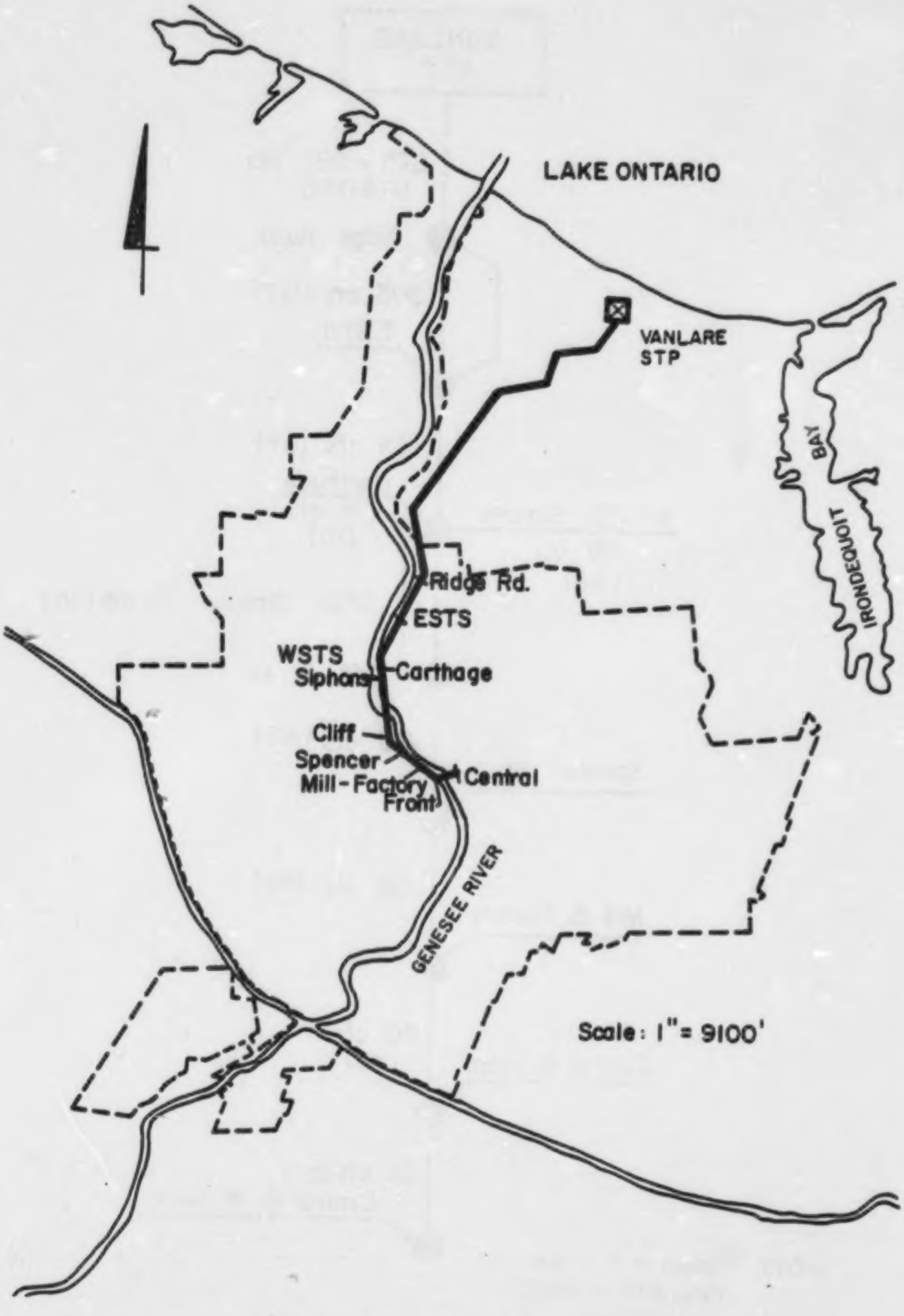

LAKE ONTARIO
VANLARE STP
IRONDEQUOIT BAY
Ridge Rd.
ESTS
WSTS
Siphons
Carthage
Cliff
Spencer
Mill-Factory
Front
Central
GENESEE RIVER
Scale: 1" = 9100'

Figure 7. Location map for St. Paul Boulevard Interceptor.

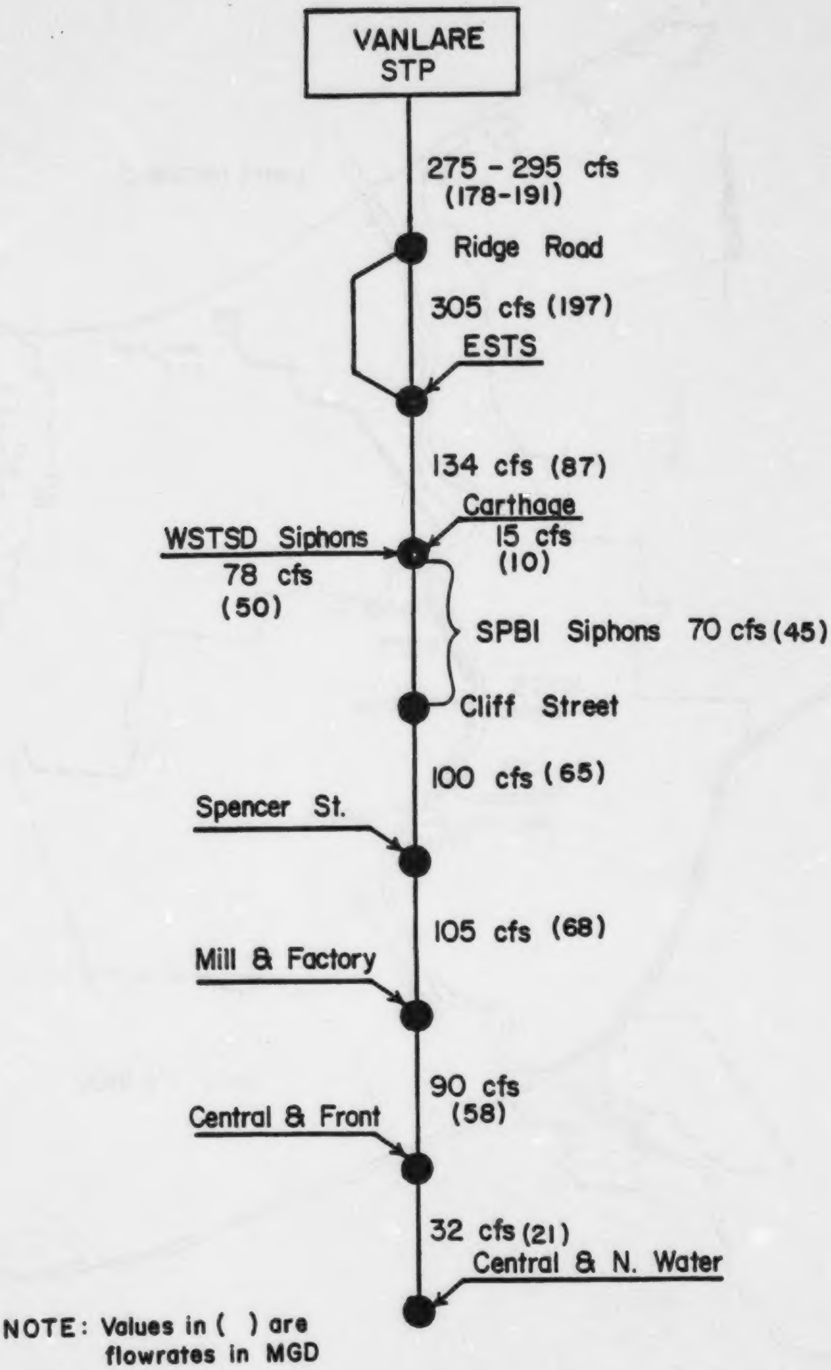

VANLARE STP
275 - 295 cfs (178-191)
Ridge Road
305 cfs (197)
ESTS
134 cfs (87)
Carthage
15 cfs (10)
WSTSD Siphons
78 cfs (50)
SPBI Siphons 70 cfs (45)
Cliff Street
100 cfs (65)
Spencer St.
105 cfs (68)
Mill & Factory
90 cfs (58)
Central & Front
32 cfs (21)
Central & N. Water
NOTE: Values in ( ) are flowrates in MGD

Figure 8. Schematic of the SPBI and trunk sewer system.

The Genesee Valley Interceptor Southwest tunnel, located in the southwestern portion of the city, was placed into operation. Flow within this system discharges to the Clarissa Street Tunnel, then to the Main and Front Street tunnels, and finally to the SPBI via the Front Street regulator. Being a large tunnel complex, a substantial amount of potential in-system storage is available.

The last major sewer in the overall collection system is the Cross-Irondequoit Tunnel (CIT) which runs from the southern end of Irondequoit Bay to the Cross-Irondequoit Pump Station (CIPS) adjacent to the VanLare STP. Wastewater must be pumped to the VanLare Treatment Plant from the CIP. Figure 6 shows the location of the tunnel and pump station. The CIT is essentially a conveyance tunnel which carries dry-weather flow from the southeastern portion of Monroe County. In the future, this tunnel will convey a portion of the CSO from the ESTS.

Within the overall wastewater collection system, there are ten grit chambers. Figure 9 shows the location of these structures. Only three of these chambers are on the combined sewer system, the remainder being located on storm sewers. Of the three on combined sewers, two are located immediately prior to siphons and one upstream of a pump station. Presently, maintenance problems are associated with these structures because of the size of the access manhole.

Currently, all overflow regulators are static devices, whereby the rate of wastewater transfer from trunk sewer to the SPBI is determined by depth of sewage flow. There is no active control such that the transfer rate can be regulated or controlled. As a result, maximum utilization of the existing sewer system cannot be realized. If additional flow could be transferred to the SPBI by means of regulator adjustments, CSO discharged to the Genesee River would be reduced.

In addition to the under-utilization of SPBI, a substantial volume of in-system storage would be potentially available, if control structures at selected locations were installed. Because the regulator and weir structures are static devices that are activated by wastewater levels in the trunk sewers, hydraulic relief must be provided to insure that adverse backwater and surcharge conditions do not develop during storm events. This mode of operation does not allow for full utilization of in-system storage. If active system control were provided, greater utilization of potential in-system storage would be realized and overflows would be reduced.

WATER QUALITY CONSIDERATIONS

The primary objective of any pollution abatement program is the improvement in receiving water quality. Three main receiving water bodies were considered in the BMP program: Lake Ontario, Genesee River, and Irondequoit Bay. A brief description of the current problems associated with each water body and the expected impacts due to CSO's follows.

The portion of the Genesee River, as it passes through the City of Rochester, is classified 'B' by the New York State Department of Environ-

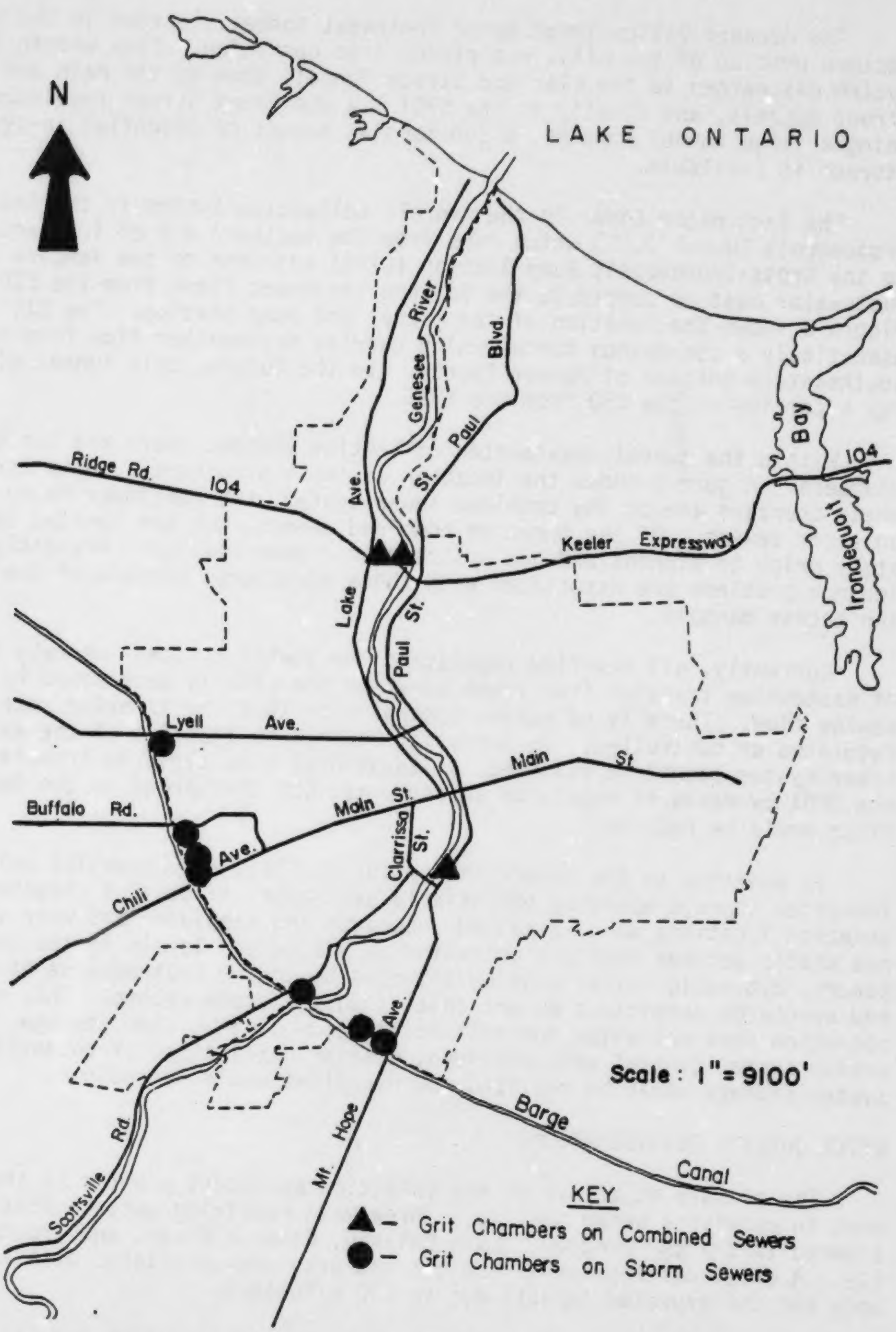

N
LAKE ONTARIO
Genesee River
Blvd.
St. Paul
Lake Ave.
Bay
104
Ridge Rd.
104
Keeler Expressway
Irondequoit
Lake
Paul St.
Lyell Ave.
Main St.
Buffalo Rd.
Main St.
Clarrissa St.
Ave.
Chili
Ave.
Scale: 1" = 9100'
Barge
Canal
Mt. Hope
Rd.
Scottsville
KEY
▲ – Grit Chambers on Combined Sewers
● – Grit Chambers on Storm Sewers

Figure 9. Grit chamber locations.

mental Conservation. This classification indicates that this water body is suitable for all uses including contact recreation but not as a source of potable water. Past studies have shown that CSO's from the existing wastewater collection system within the city result in contraventions of established stream standards (1,2,6). The most significant impacts are depressed dissolved oxygen concentrations and high fecal coliform levels.

Based on previous mathematical simulations of receiving water quality for the Genesee River, under varying flow and loading conditions, the following conclusions were formulated (2):

1. The pollutional impact of CSO's causes the contravention of established water quality standards for the Genesee River.

2. Abatement of CSO discharges and adequate treatment from continuous discharges from the existing Gates-Chili-Ogden and Kodak waste treatment facilities are required to meet the water quality standards of the Genesee River.

3. CSO treatment efficiencies in the range of 90% to 100% are required to maintain the dissolved oxygen standards of the river during summer low flow periods.

A major concern over the past several years has been the pollution problems associated with the Rochester Embayment of Lake Ontario. The Rochester Embayment is that portion of Lake Ontario immediately adjacent to the City of Rochester. Extending approximately eight miles out from shore at the mouth of the Genesee River, the Embayment is nearly 30 mi across. Figure 1 illustrates the area defined as the Embayment. During periods of rainfall, the current CSO discharges to the Genesee River result in high fecal coliform levels in the Embayment. Because there are public beaches in this area, the high bacteria levels have forced the Monroe County Health Department to close these beaches until acceptable bacterial levels are reached, usually several days after an intense storm event.

Lake Ontario and specifically the Rochester Embaymant are classified as 'A-Special' by the New York State Department of Environmental Conservation. Under this classification best usage is defined as water suitable for drinking, food processing purposes, and contact recreation. Previous water quality modeling of the Genesee River demonstrated that fecal coliform levels in excess of established standards occur during storm events (2).

Irondequoit Bay is a prime water resource of Monroe County in terms of recreational potential. It is a relatively shallow body of water that is bordered by low-lying urban areas. The eastern portion of the City of Rochester and much of the southeastern portion of Monroe County drain to the bay and have contributed to its progressive eutrophication. A comprehensive sewerage study was made in the late 1960's to define a water quality management program necessary for the enhancement of Irondequoit Bay (8). The program which was adopted, and is now being implemented, requires complete diversion from the bay of all wastewater and CSO discharges. Extensive limnological studies of the bay ecosystem were also conducted between 1969 and 1976 (10,11,12,13). These studies provided the necessary data to evaluate the impact of the diversion program in reversing the progressive eutrophication of the bay.

All of the previous studies clearly demonstrated that under current conditions the bay is nutrient limited. This indicates that, although eutrophication is now impairing the bay's intended best usage, extensive reduction in phosphorus loading may be expected to result in a significant restoration of water quality.

The water quality objectives for the three main water bodies impacted by CSO's from the City of Rochester are summarized in Table 5.

TABLE 5. WATER QUALITY OBJECTIVES

RECEIVING WATER BODY	PRIMARY OBJECTIVE
Genesee River	Maintenance of the dissolved oxygen standard--minimum 4.0 mg/l; average daily 5.0 mg/l
Rochester Embayment of Lake Ontario	Maintenance of the established fecal coliform standard--200 counts/100 ml
Irondequoit Bay	Maximum reduction of nutrient loadings

SECTION 5

OVERFLOW MONITORING

GENERAL

To establish the impact of CSO discharges on the Genesee River and to evaluate the effectiveness of implemented abatement measures, accurate and reliable overflow monitoring is essential. Integral to this determination is flow monitoring to establish the rate and volume of CSO discharged during storm events as well as sampling and analytical analysis to determine the pollutional characteristics of the overflow. Knowing both flows and pollutant concentrations, total pollutant loadings to the receiving waters can be computed. It is important to also establish the relationship between rainfall and overflow volume. Therefore, an accurate rainfall monitoring program is necessary in addition to overflow monitoring and sampling.

Flow Monitoring

During the BMP Program nine overflow sites, identified in Table 6, were monitored to determine the rate and volume of CSO discharged during storm events. The location of each site is shown in Figure 10. Except for the Spencer location, each of the identified sites had been monitored under a previous USEPA R&D grant (1). A review of the operational status of the original CSO monitoring equipment was conducted to determine the necessary work required to provide satisfactory flow monitoring at each overflow site. Much of the equipment utilized during that program had to be replaced with more reliable equipment.

In terms of field installation, an ultrasonic or bubbler level measuring and recording system was installed within the actual overflow conduit at each overflow location. The primary measurement, level of wastewater, was converted to a flow value using either an open-channel, Manning equation or a weir equation. Figures 11 and 12 show a typical head measurement system for an open-channel and a weir flow determination, respectively. Table 7 summarizes by site the type of equation used to determine flowrate from the primary measurement.

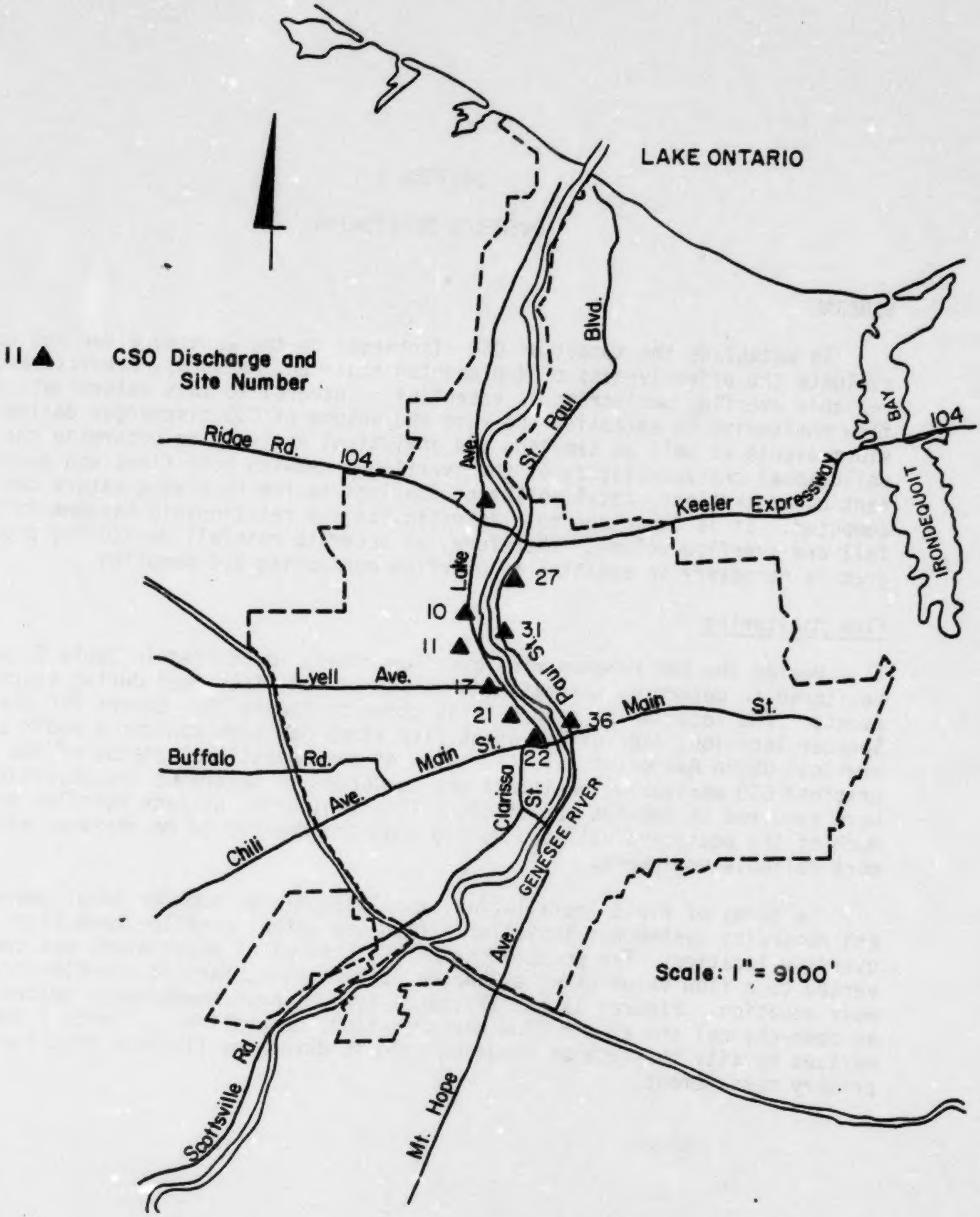

LAKE ONTARIO
11 ▲ CSO Discharge and Site Number
Ridge Rd.
104
Lake Ave.
St. Paul Blvd.
BAY
104
IRONDEQUOIT
Keeler Expressway
7
27
10
31
11
Lake
St. Paul St.
Lyell Ave.
17
21
36
Main St.
22
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
GENESEE RIVER
Scale: 1"= 9100'
Mt. Hope Ave.
Scottsville Rd.

Figure 10. CSO monitored sites within the City of Rochester.

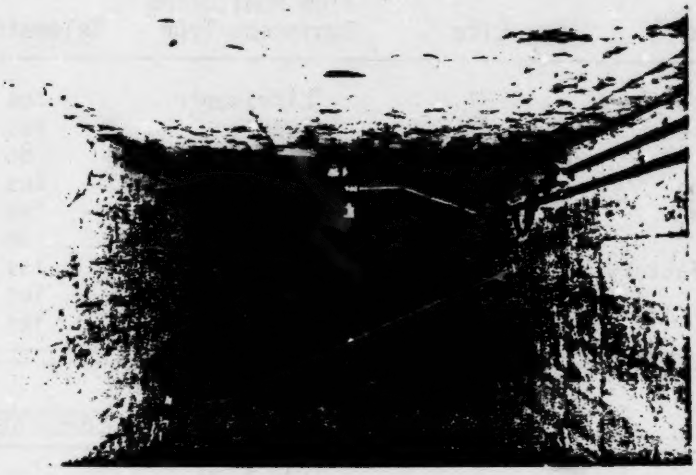


Figure 11. Head measurement for open-channel monitoring location

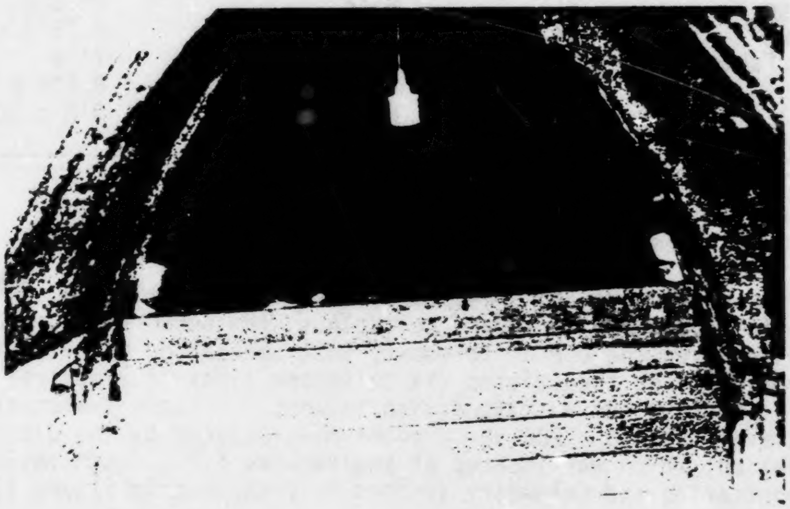


Figure 12. Head measurement for weir monitoring location

TABLE 6. OVERFLOW MONITORING INSTALLATIONS

Location	Site	Flow Monitoring Equipment-Type	Telemetry	Sampling
Maplewood	7	Ultrasonic	Yes	Yes
Seth Green	27	Bubbler	Yes	Yes
WSTS	11	Ultrasonic	No	Yes
Lexington	10	Ultrasonic	Yes	Yes
Carthage	31	Ultrasonic	Yes	Yes
Spencer	17	Ultrasonic	No	No
Mill & Factory	21	Bubbler	Yes	Yes
Front	22	Ultrasonic	Yes	Yes
Central	36	Ultrasonic	Yes	Yes

TABLE 7. CHARACTERISTIC FLOW EQUATIONS

Location	Site	Type Flow Equation	Size and Shape of Conduit*
Maplewood	7	Open Channel	5'H x 6'W Rect.
Seth Green	27	Open Channel	6.83' ø
WSTS	11	Open Channel	7.92' x 8.5' H.S.
Lexington	10	Weir	7' S.E.
Carthage	31	Weir	7' S.E.
Spencer	17	Weir	4.75'H x 3.5'W B.H.
Mill & Factory	21	Weir	7' ø
Front	22	Weir	8.5'H x 8.17'W B.H.
Central	36	Weir	9'H x 10.5'W B.H.

*Rect. = Rectangular, ø = circular, H.S. = Horseshoe,
S.E. = Semi-elliptical, B.H. = Basket-handle

Primary and secondary recording systems were utilized at most overflow monitoring locations to insure reliable system performance. The primary system involved the use of telemetry instrumentation to allow the site measurement to be transmitted via telephone lines to a central receiving center. Those sites outfitted with telemetry systems transmitted the monitored level data to a PDP-8E computer manufactured by the Digital Equipment Corporation, which was located at the VanLare STP. A schematic of the overflow monitoring and telemetry systems is presented in Figure 13.

The secondary or backup data recording system involved individual Rustrak recorders located at each site. During times when the telemetry system was inoperative, flow data were recorded and retained on chart paper. Without this secondary system, a significant amount of data would have been lost. A backup recording system should be installed on overflow conduits during any long-term monitoring program.

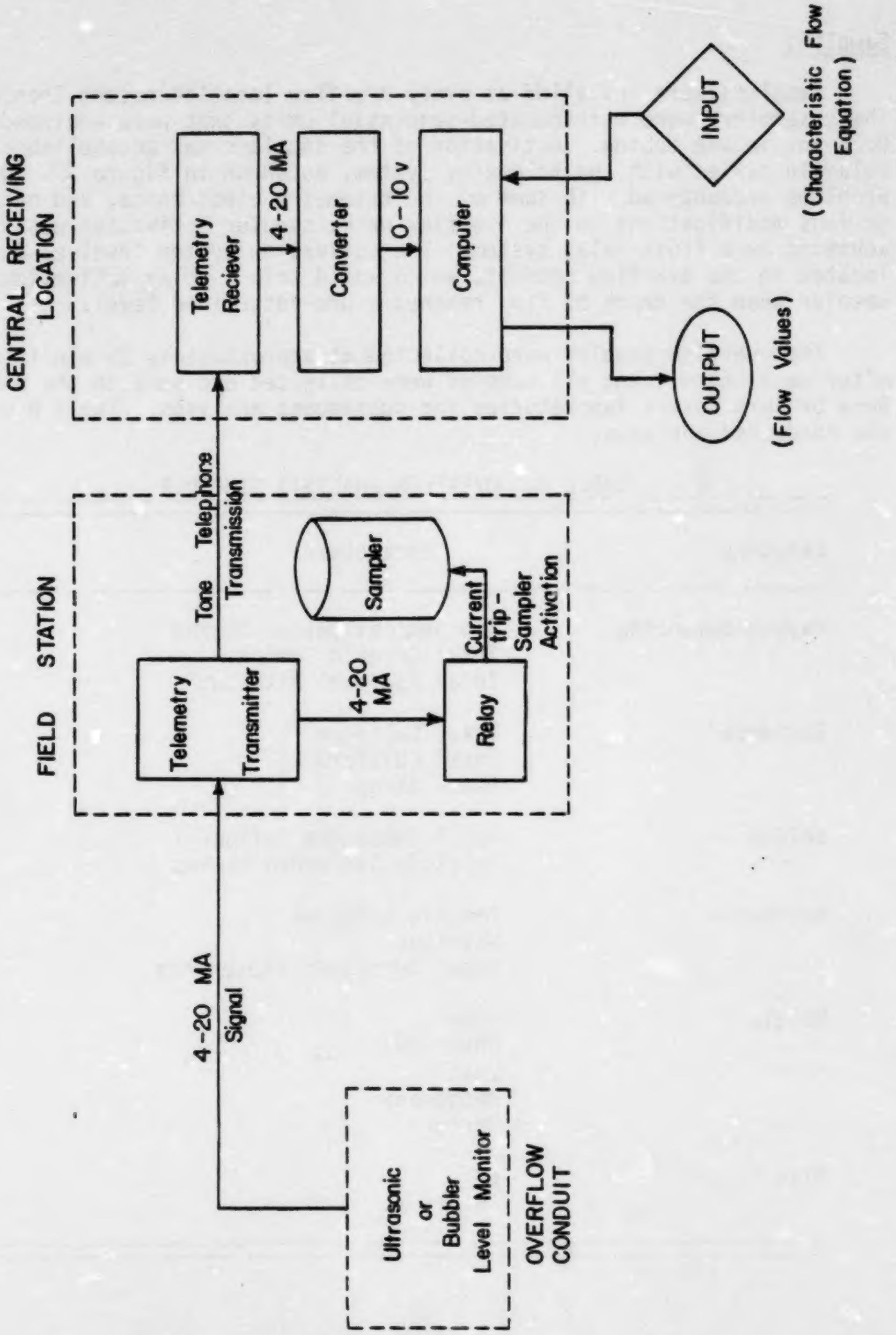

CENTRAL RECEIVING LOCATION
Telemetry Reciever
4-20 MA
Converter
0-10 v
Computer
INPUT
(Characteristic Flow Equation)
OUTPUT
(Flow Values)
FIELD STATION
Tone Telephone Transmission
Telemetry Transmitter
4-20 MA
Relay
Sampler
Current trip-Sampler Activation
4-20 MA Signal
Ultrasonic or Bubbler Level Monitor
OVERFLOW CONDUIT

Figure 13. Schematic of overflow monitoring and telemetry systems.

Sampling

Samplers were installed at every overflow location except Spencer. These samplers were refrigerated-sequential units that were equipped with 0.25 in. intake tubing. Activation of the samplers was accomplished by a relay in series with the telemetry system, as shown in Figure 13. Due to problems encountered with some of the telemetry electronics, and because of various modifications to the overflow weir, sampler activation was primarily achieved by a float-relay system. The activation system involved a float located in the overflow conduit, which would trip a relay activating the sampler when the depth of flow reached a pre-determined level.

The overflow samples were collected at approximately 15 min intervals. After each storm event all samples were collected and sent to the O'Brien & Gere or Pure Waters laboratories for subsequent analyses. Table 8 presents the conducted analyses.

TABLE 8. OVERFLOW ANALYSIS SCHEDULE

Category	Parameters
Oxygen-demanding	Biochemical Oxygen Demand Total Organic Carbon Total Kjeldahl Nitrogen
Bacteria	Total Coliform Fecal Coliform Fecal Strep
Solids	Total Suspended Solids Volatile Suspended Solids
Nutrients	Ammonia Nitrogen Nitrites Total Inorganic Phosphorus
Metals	Iron Chromium Lead Manganese Mercury
Misc.	pH Chlorides

The purpose of the overflow monitoring and sampling program was fourfold:

(1) To determine whether any sewer changes upstream of the overflow diversion point had caused significant changes in volume and frequency of overflow relative to that observed under the former USEPA R&D program.

(2) To determine whether the pollutant concentrations previously measured during the 1973-1975 period were reasonably similar to those determined under the present program.

(3) To determine whether the sites that had previously exhibited a first-flush condition continued to do so under the BMP program, and also whether those sites exhibiting the highest pollutant loadings - termed high-impacting sites - still showed similar rankings.

(4) To provide for reliable and accurate overflow quantity and quality determinations to allow for a proper assessment of effectiveness of the scheduled regulator and weir modifications.

The primary objective of the flow monitoring program was not to evaluate the performance of various types of equipment and instrumentation as was done under the previous R&D grant (1). The essential objective was the collection of reliable, accurate flow data to satisfactorily evaluate the effectiveness of the BMP program.

Often overlooked in many previous CSO projects throughout the country is the need for installing adequate flow monitoring equipment and properly maintaining that instrumentation throughout the duration of the project. It cannot be over-emphasized that without accurate and reliable flow monitoring equipment, no evaluations can be conducted. An intense effort was made throughout the BMP program to provide each overflow site with adequate flow monitoring and sampling equipment and to maintain all units to insure satisfactory data collection during storm events.

In addition to insuring that adequate equipment was installed and maintained, it was important to determine the most appropriate flow equation to be used in translating the wastewater depth reading recorded by the equipment into a flowrate. As shown in Table 7, the particular flow equation used was a weir formulation for determining overflow rates at most sites. In three instances, the most appropriate equation was a Manning formula applied under the assumption that open-channel flow conditions occurred in the sewer system.

To highlight some of the results of the overflow monitoring program conducted under the BMP program, the remainder of Section 5 is subdivided into subsections on rainfall, flow monitoring, and quality considerations. As previously indicated, it was not the purpose of the BMP program to establish an overflow monitoring program, but only to use the collected data in assessing the effectiveness of all proposed and implemented abatement control measures.

RAINFALL ANALYSIS

One of the most important factors in the generation of storm-induced CSO volumes and pollutant loadings is rainfall. All methods for estimating these storm-induced loadings rely on rainfall data as fundamental input for reasonable and meaningful projections. Much effort was expended in the area of network modeling and the collection of CSO monitoring and sampling data. An important element of the total effort was the accurate and meaningful development of rainfall hyetographs for various return period frequencies from Weather Bureau frequency-intensity-duration curves or actual local rainfall data, which was then utilized for CSO loading projections.

Daily and hourly precipitation records from the U.S. Weather Bureau station at the Monroe County Airport were used in evaluating BMP options. Rainfall records, covering a period from 1954 to 1975, were obtained from the National Climate Center, Asheville, North Carolina. The data indicated that precipitation patterns and durations for the Rochester area were highly variable. High-intensity, short-duration events were usually associated with thunderstorms which occurred during the summer months; whereas, low intensity, long-duration events were usually associated with cyclonic activity which occurred during the spring and fall months.

Table 9 summarizes the monthly and annual rainfall statistics based on 22 yr of precipitation records collected during the period of January, 1954 to December, 1975. In addition, Tables 10 and 11 summarize the number of days with precipitation and the rain per day associated with the period of record.

TABLE 9. AVERAGE MONTHLY RAINFALL IN THE ROCHESTER AREA - 1954 to 1975

Month	Average Rainfall, in.	Standard Deviation	Coefficient of Variation	95% Confidence Level Interval±
Jan	2.11	0.87	0.41	0.39
Feb	2.45	1.10	0.45	0.49
Mar	2.36	1.01	0.43	0.45
Apr	2.58	0.83	0.32	0.37
May	2.65	1.20	0.45	0.53
Jun	2.81	1.61	0.57	0.71
Jul	2.30	1.23	0.54	0.55
Aug	3.31	1.26	0.38	0.56
Sep	2.29	1.14	0.50	0.51
Oct	2.52	1.81	0.72	0.80
Nov	2.76	1.12	0.41	0.50
Dec	2.50	1.10	0.44	0.49
ANNUAL	30.63	4.43	0.14	1.97

TABLE 10. AVERAGE MONTHLY NUMBER OF RAINDAYS IN THE ROCHESTER AREA - 1954 to 1975

Month	Average No. of Raindays	Standard Deviation	Coefficient of Variation	95% Confidence Level of Interval±
Jan	15.64	3.16	0.20	1.40
Feb	14.77	3.70	0.25	1.64
Mar	13.82	3.62	0.26	1.61
Apr	12.91	2.74	0.21	1.22
May	11.59	3.42	0.29	1.52
Jun	9.41	3.22	0.34	1.43
Jul	9.23	3.12	0.34	1.38
Aug	10.18	2.13	0.21	0.94
Sep	10.00	3.80	0.38	1.69
Oct	11.05	3.12	0.28	1.39
Nov	15.18	2.92	0.19	1.30
Dec	17.73	3.15	0.18	1.40
ANNUAL	151.50	12.91	0.09	5.73

TABLE 11. AVERAGE MONTHLY RAIN PER STORM IN THE ROCHESTER AREA - 1954 to 1975

Month	Average Rain Per Storm in.	Standard Deviation	Coefficient of Variation	95% Confidence Level Interval±
Jan	0.14	0.05	0.40	0.20
Feb	0.17	0.07	0.40	0.03
Mar	0.17	0.05	0.31	0.02
Apr	0.21	0.07	0.33	0.03
May	0.23	0.09	0.37	0.04
Jun	0.29	0.12	0.41	0.05
Jul	0.26	0.13	0.51	0.06
Aug	0.33	0.11	0.33	0.05
Sep	0.23	0.10	0.43	0.04
Oct	0.21	0.12	0.55	0.05
Nov	0.18	0.06	0.34	0.03
Dec	0.14	0.06	0.43	0.03
ANNUAL	0.21	0.02	0.11	0.01

From Tables 9 through 11, it can be seen that monthly precipitation in Rochester was fairly uniform throughout the year. As expected, however, the months of June, July and August exhibited the higher intensity, shorter duration storm events. During the same period of record, the greatest total amount of rainfall presented by a storm event was 3.91 in. over 87 hr. The greatest maximum hourly intensity was 1.35 in. with a duration of 8 hr and a total rainfall depth of 2.15 in.

A rainfall characterization routine was utilized to define discrete storm events and rank design parameters associated with each storm. The input data to this program was the hourly rainfall record from the U.S. Weather Bureau. For this study, a discrete storm event was defined as starting with the first measurable rainfall after a minimum interval of 6 hr with no rainfall and ending when a gap in measured rainfall of at least 6 hr was first encountered. For each event in the historical record, the following parameters were calculated: date, starting hour, duration, total rainfall, the elapsed time from the previous storm, snowfall, and the ratio of the hour of maximum rainfall to total duration (r value). Figures 14 through 17 are examples of frequency curves based on ranked rainfall data as provided by the program. Storm parameters can be determined from these curves. The validity and usefulness of these curves was directly related to the length of the available record. Figure 18 presents the rainfall intensity-frequency-duration curves for Rochester as determined from the U.S. Weather Service (14).

Tables 12 and 13 summarize rainfall data in the Rochester area for the BMP monitoring period from January 1979 to August 1980. An examination of data for 1979 indicated that 1979 was slightly higher in total precipitation than an average year, although March, June, August and November were relatively dry months. Of significance was that over one-third of the total annual precipitation in 1979 was accounted for by the 12 storms indicated. A storm which occurred on September 13-14 accounted for 3.54 in. of the monthly total precipitation of 5.32 in. Examination of the 1980 precipitation data showed that 1980 had approximately the same amount of precipitation as an average year, even though 6 of the 8 months had below average precipitation. This was further highlighted by the quantities shown in Table 14. The month of June contributed over 20 percent of the total precipitation through August as a result of two large storms which occurred back-to-back on June 6 and 7, both of relatively short duration considering the total rainfall depth. Such events, as indicated in Tables 12 and 13, were anticipated to have major impacts on the conveyance network and contribute greatly to the discharge of CSO.

In subsequent modeling of the conveyance system for the City of Rochester with the Simplified Stormwater Model (SSM), an average rainfall year was used. That is, a particular year (1975) was selected that had a total annual precipitation amount approximately equal to the average annual volume.

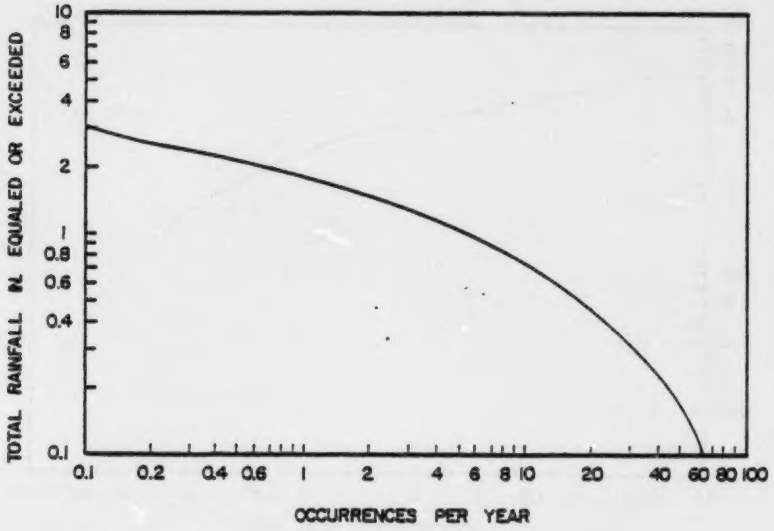


Figure 14. Example curve - storm magnitude vs. frequency.

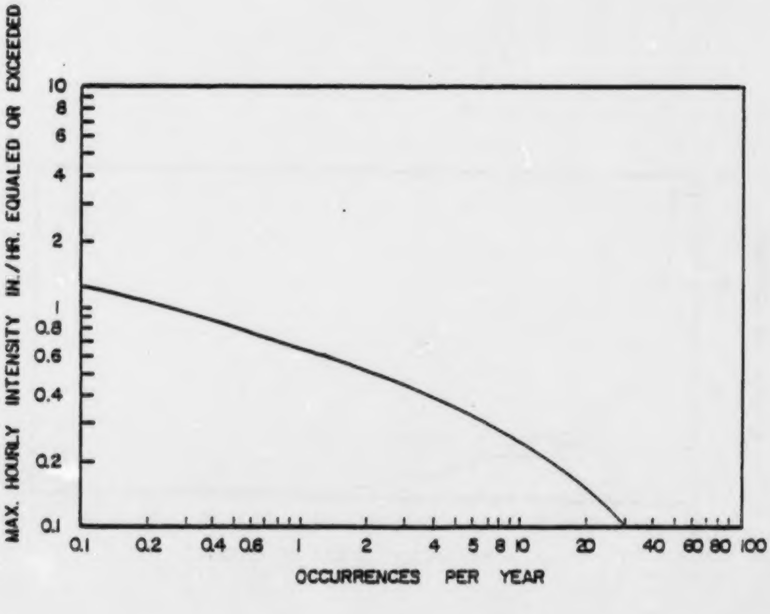


Figure 15. Example curve - storm intensity vs. frequency.

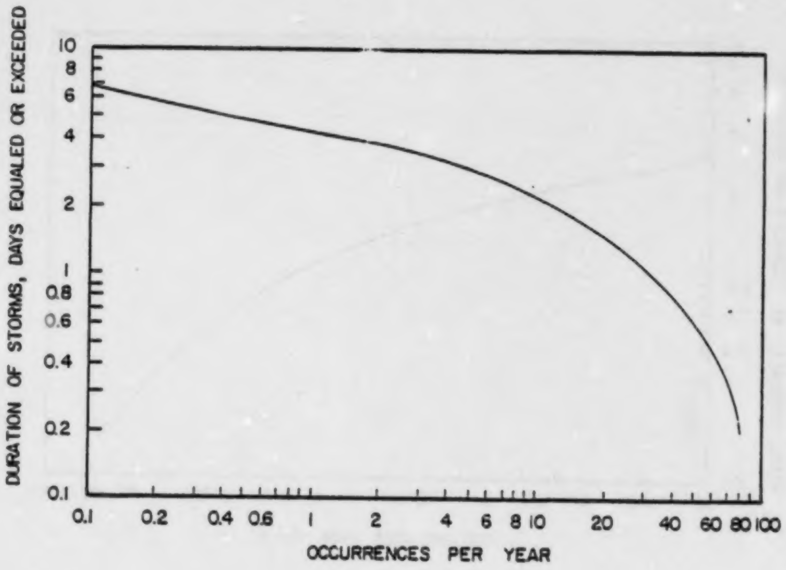


Figure 16. Example curve - storm duration vs. frequency.

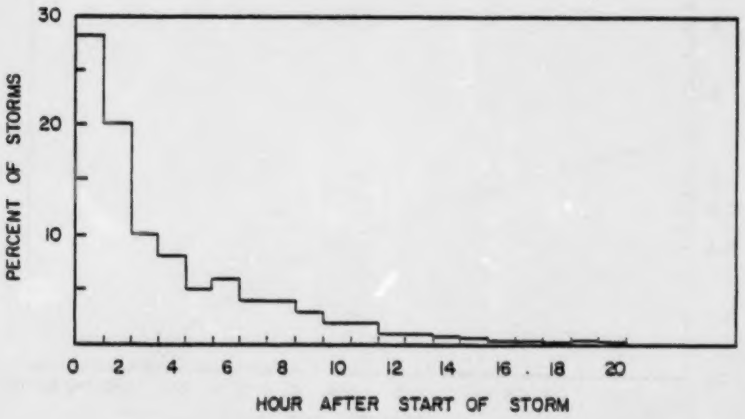
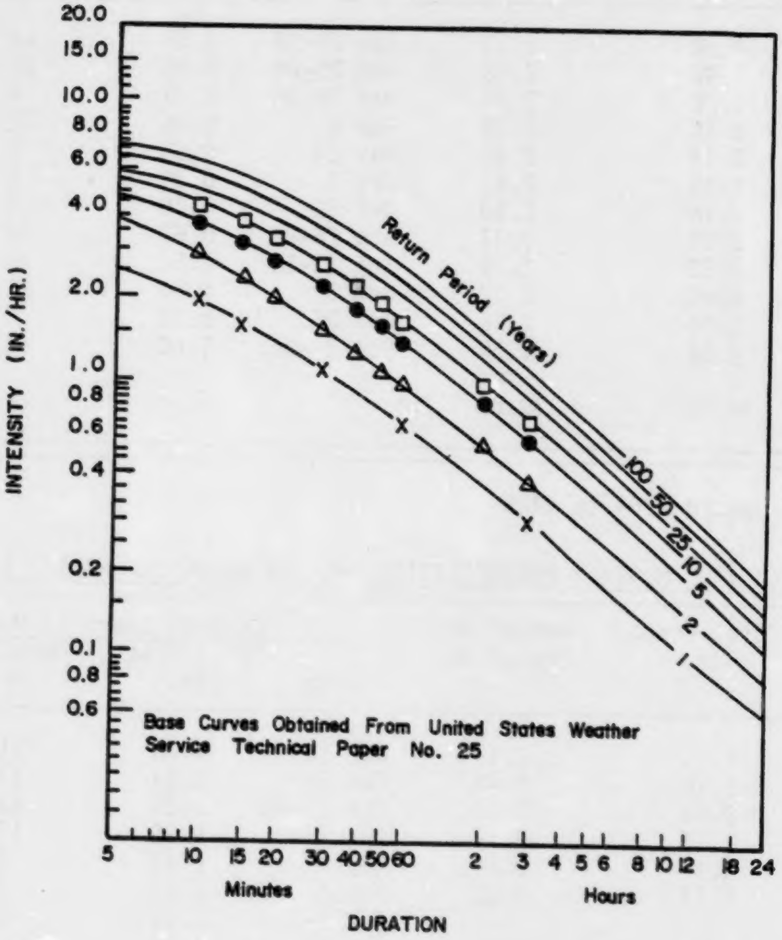


Figure 17. Example curve - percent of storms having maximum hourly intensity vs. hour after start of storm.



Base Curves Obtained From United States Weather Service Technical Paper No. 25

Figure 18. Rainfall frequency-intensity-duration curves for Rochester, New York.

TABLE 12. MONTHLY PRECIPITATION DATA IN ROCHESTER AREA - 1979

Month	Total Prec. in.	Annual Avg. Prec.* in.	Largest Storm in Month			
			Date	Total Prec. in.	Duration hrs	Snow Incl
Jan	4.18	2.11	Jan 24-26	1.33	42	Yes
Feb	2.40	2.45	Feb 25-26	0.90	21	Yes
Mar	1.76	2.36	Mar 29-30	0.37	6	No
Apr	3.78	2.58	Apr 6	0.86	21	Yes
May	3.14	2.65	May 24	0.63	6	No
Jun	1.85	2.81	Jun 7	0.34	2	No
Jul	3.16	2.30	Jul 31	0.70	4	No
Aug	2.05	3.31	Aug 26-27	0.63	12	No
Sep	5.32	2.29	Sep 13-14	3.54	16	No
Oct	2.60	2.52	Oct 5	0.84	21	No
Nov	1.80	2.76	Nov 26	0.38	7	No
Dec	2.86	2.50	Dec 24-26	1.60	44	No
ANNUAL	34.90	30.63		12.12		

*Period of Record 1954 to 1975

TABLE 13. MONTHLY PRECIPITATION DATA IN ROCHESTER AREA - 1980

Month	Total Prec. in.	Annual Avg. Prec.* in.	Largest Storm in Month			
			Date	Total Prec. in.	Duration hrs	Snow Incl
Jan	1.11	2.11	Jan 11	0.38	11	No
Feb	1.16	2.45	Feb 16	0.34	17	Yes
Mar	3.83	2.36	Mar 21-22	1.20	43	Yes
Apr	2.35	2.58	Apr 28	0.99	12	No
May	1.49	2.65	May 31	0.47	2	No
Jun	6.77	2.81	Jun 6	2.19	4	No
			Jun 7-8	2.16	3	No
Jul	1.90	2.30	Jul 22	1.03	16	No
Aug	2.68	3.31	Aug 5	0.72	1	No
			Aug 5-6	0.60	6	No
ANNUAL (to Aug)	21.29	20.57		10.08		

*Period of record 1954 to 1975

TABLE 14. COMPARISON OF ANNUAL AVERAGE RAINFALL DATA VERSUS 1980 RAINFALL DATA

Month	Avg. Rain, in.	Avg. Rain Days/Mo.*	Avg. Rain Per Rain Day	Standard Dev, in.	1980 Rain, in.	No. Rain Days/Mo.	Avg. Rain Per Rain Day, in.
Jan	2.11	16	0.13	0.87	1.11	15	0.07
Feb	2.45	15	0.16	1.10	1.16	18	0.06
Mar	2.36	14	0.17	1.01	3.83	16	0.24
Apr	2.58	13	0.20	0.83	2.35	14	0.17
May	2.65	12	0.22	1.20	1.49	9	0.17
Jun	2.81	9	0.31	1.61	6.77	15	0.45
Jul	2.30	9	0.26	1.23	1.90	10	0.19
Aug	3.31	10	0.33	1.26	3.22	10	0.32
Σ	20.57				21.83		

* No. of days per month in which precipitation (water equivalent) was $\geq$ 0.01 in.

In addition to the Weather Bureau precipitation data, local rain gauges collected rainfall data that was subsequently used in the overflow monitoring analysis. The Weather Bureau data base served to assess the effectiveness of implemented BMP measures on an average basis. With respect to overflow monitoring, however, the actual CSO discharge volumes and rates from the various drainage areas were directly related to site-specific rainfall intensities and volumes. If rainfall within the City of Rochester was not uniform, then an adjustment was made to correlate rainfall and overflow discharge characteristics on a site by site basis.

To determine the applicability of utilizing Weather Bureau data to assess the frequency and volume of CSO from the drainage basins, a system of 8 rain gauges had been installed within the Rochester Pure Waters District under a previous combined sewer overflow monitoring and sampling program (1). Table 15 identifies the rain gauge locations and Figure 19 depicts their location within the District. All rain gauges were Fischer & Porter bucket-type instruments, Model No. 35B1558. These gauges recorded the cumulative rainfall at five-minute intervals on punch paper tape for later transfer to computer magnetic tape. The gauges were designed to initiate data collection and provide transfer to punch tape in rainfall increments of 0.10 in.

TABLE 15. LOCAL RAIN GAUGES

Location	Site	Distance from Weather Bureau Gauge at Airport-mi
East High School	RO-3	6.36
Marshall High School	RO-4	5.86
School #44	RO-5	2.22
Brighton Middle School	RO-6	5.72
Fire Department Headquarters	RO-7	4.71
Norton Densmore Chlorination Station	RO-8	8.01
Franklin High School	RO-10	6.43
Charlotte Pump Station	RO-11	9.90

Operation of this network of rain gauges was continued under the BMP program. However, one significant modification to the data reporting process was made. The punch tape mechanism was removed from each rain gauge to permit the installation of an electrical device to allow direct telemetering of rainfall data to the centralized computer facilities at the VanLare STP. Athough the processing of data was thereby expedited, backup local data recorders were sacrificed. The lack of local recorded data can prove to be detrimental to a rainfall data collection program. If for any reason the telemetry system did not operate satisfactorily, no useful rainfall data would be obtained. Local recording devices could avoid problems caused by telemetry malfunctions. For most storm events, however, the local rain gauges and the associated telemetry system performed adequately.

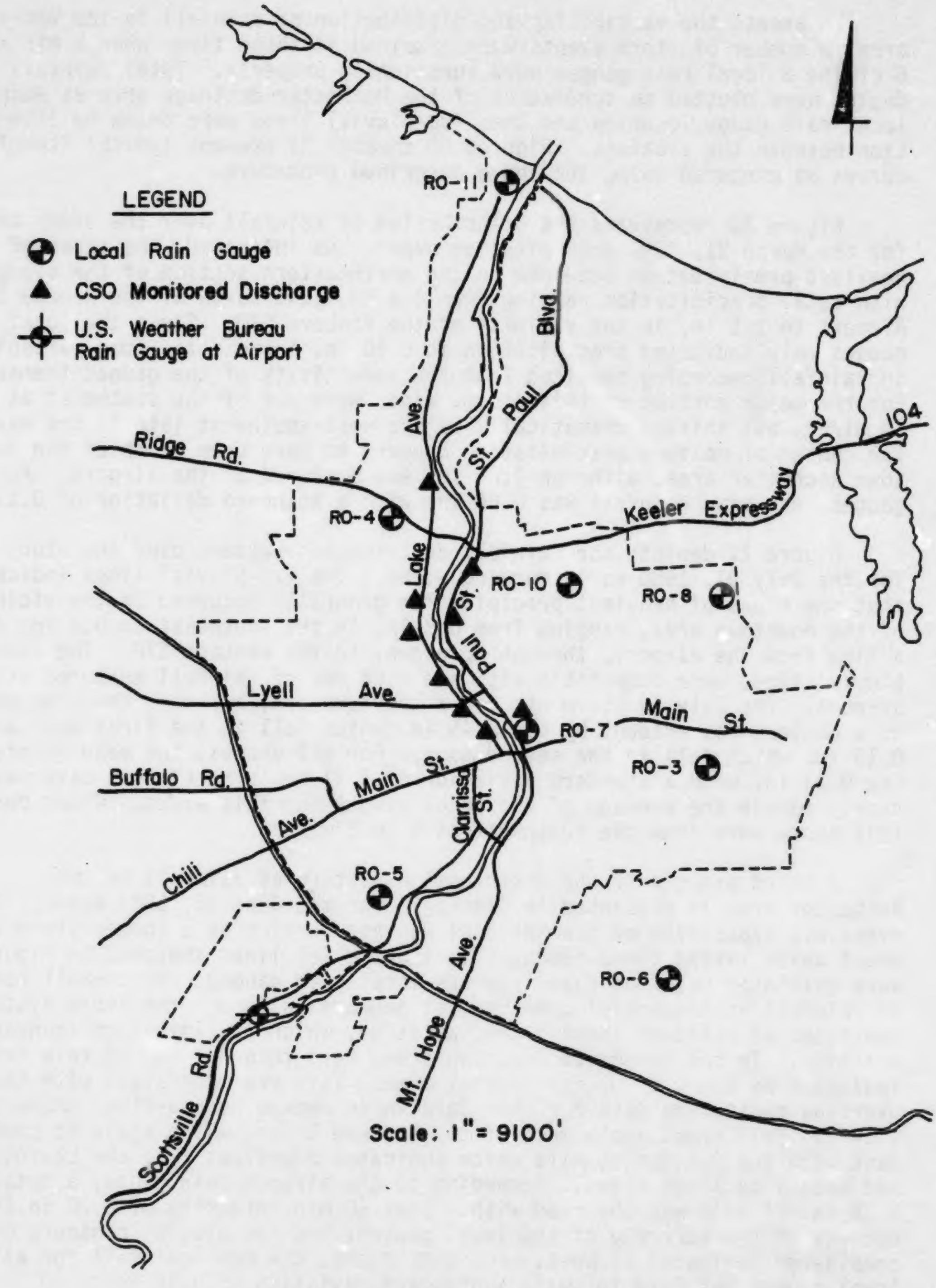

LEGEND
Local Rain Gauge
CSO Monitored Discharge
U.S. Weather Bureau Rain Gauge at Airport
RO-11
RO-4
RO-10
RO-8
RO-7
RO-3
RO-5
RO-6
Ridge Rd.
Lake Ave.
St. Paul Blvd.
St. Paul
104
Keeler Expressway
Lyell Ave.
Main St.
Buffalo Rd.
Chili Ave.
Clarissa St.
Mt. Hope Ave.
Scottsville Rd.
Scale: 1" = 9100'

Figure 19. Rain gauge location map.

To assess the variability and distribution of rainfall in the Rochester area, a number of storm events were examined at those times when a minimum of 6 of the 8 local rain gauges were functioning properly. Total rainfall depths were plotted on schematics of the Rochester drainage area at each local rain gauge location and then iso-pluvial lines were drawn by interpolation between the stations. Figures 20 through 22 present typical iso-pluvial curves as prepared using the above described procedure.

Figure 20 represents the distribution of rainfall over the study area for the March 21, 1980 precipitation event. As indicated, the areas of heaviest precipitation occurred in the northeastern section of the study area, with total precipitation ranging from 0.6 in. just north of the Monroe County Airport to 1.1 in. in the vicinity of the VanLare STP. Since the local gauges only indicated precipitation in 0.10 in. increments, some variability in rainfall recording resulted from the sensitivity of the gauges themselves. For the major portion of this storm, winds were out of the southeast at 10 to 15 mi/hr, but shifted dramatically to the west-southwest late in the day. The center of maximum precipitation appears to have been north of the downtown Rochester area, although 0.84 in. was measured at the airport. For all gauges, the mean rainfall was 0.84 in. with a standard deviation of 0.17 in.

Figure 21 depicts the rainfall distribution pattern over the study area for the July 22, 1980 early morning event. The iso-pluvial lines indicated that the areas of heaviest precipitation generally occurred in the vicinity of the downtown area, ranging from 0.3 in. in the southeast to 0.6 in. along a line from the airport, through downtown, to the VanLare STP. The iso-pluvial lines were compatible with the 0.58 in. of rainfall measured at the airport. The July 22 storm was not a thunderstorm; however, the high moisture content was evident by the 0.45 in. which fell in the first hour and 0.13 in. which fell in the second hour. For all gauges, the mean rainfall was 0.33 in. with a standard deviation of 0.21 in. The airport data was nearly double the average of the local gauges for this event. Winds during this storm were from the southwest at 6 to 8 mi/hr.

A third example of the distribution pattern of rainfall in the Rochester area is presented in Figure 22 for the June 26, 1980 event. This event was classified by the National Weather Service as a thunderstorm type event which lasted three hours. The iso-pluvial lines sketched in Figure 25 were estimated based on data from the local rain gauges. No overall pattern of rainfall is discernible; rather, it appears as though the storm system consisted of cells of intense precipitation, which is typical of thunderstorm activity. In the immediate downtown area, less than 0.1 in. of rain fell as indicated by the 0.0 in. iso-pluvial line. This was consistent with the overflow monitoring data for this date which showed no overflow. More than 0.10 in. fell immediately west of the Genesee River, which again is consistent with the monitoring data which indicated overflows from the Lexington and West Side Trunk sites. According to the airport rain gauge, a total of 0.28 in. of rain was observed with a peak 60 min intensity of 0.20 in./hr. Because of the accuracy of the local gauges, the iso-pluvial contours must be considered estimates at best. For this storm, the mean rainfall for all local gauges was 0.14 in. with a standard deviation of 0.12 in.

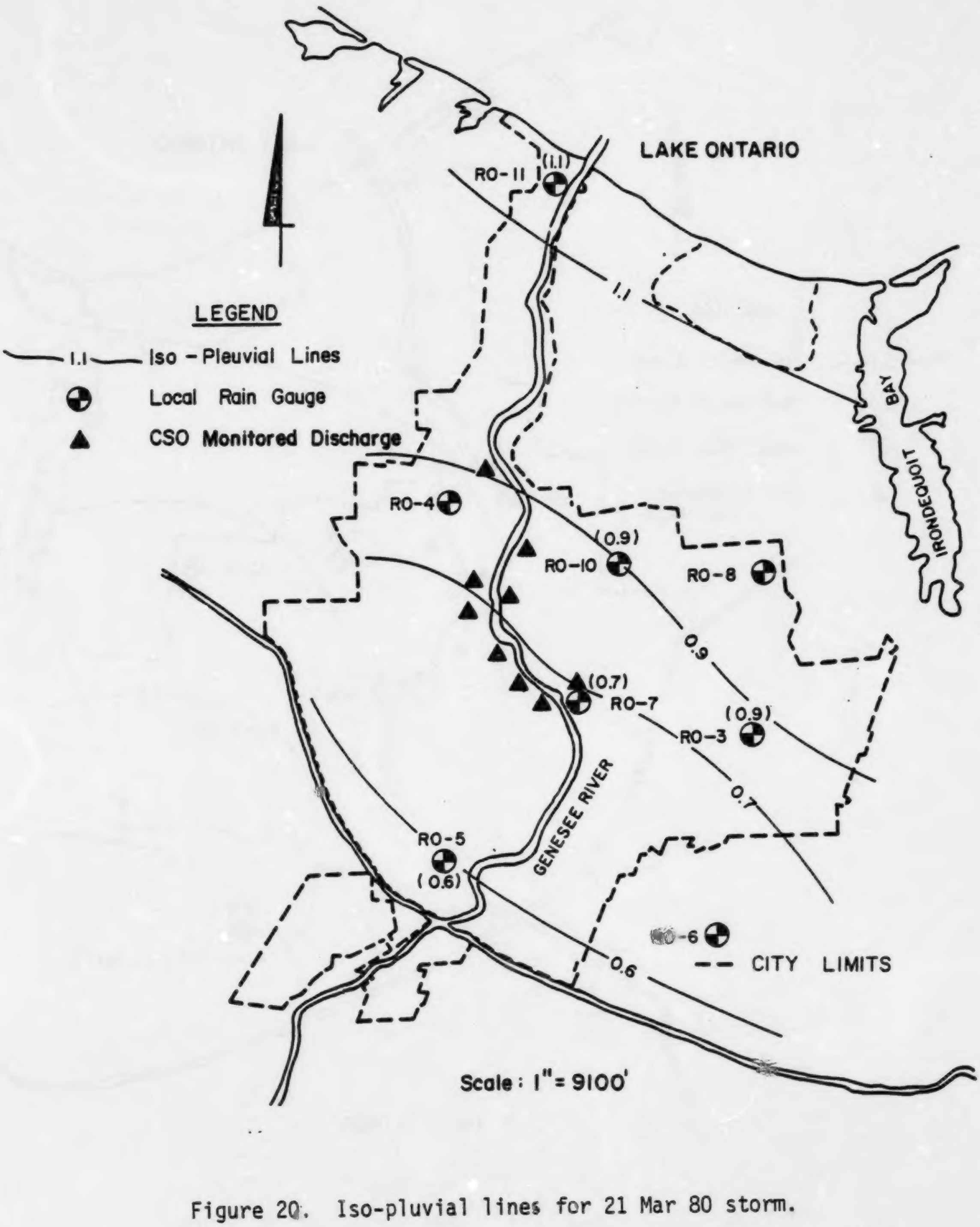

LAKE ONTARIO
RO-11
(1.1)
1.1
LEGEND
1.1 Iso - Pleuvial Lines
Local Rain Gauge
CSO Monitored Discharge
RO-4
(0.9)
RO-10
RO-8
0.9
(0.7)
RO-7
(0.9)
RO-3
0.7
IRONDEQUOIT BAY
GENESEE RIVER
RO-5
(0.6)
RO-6
0.6
CITY LIMITS
Scale: 1" = 9100'

Figure 20. Iso-pluvial lines for 21 Mar 80 storm.

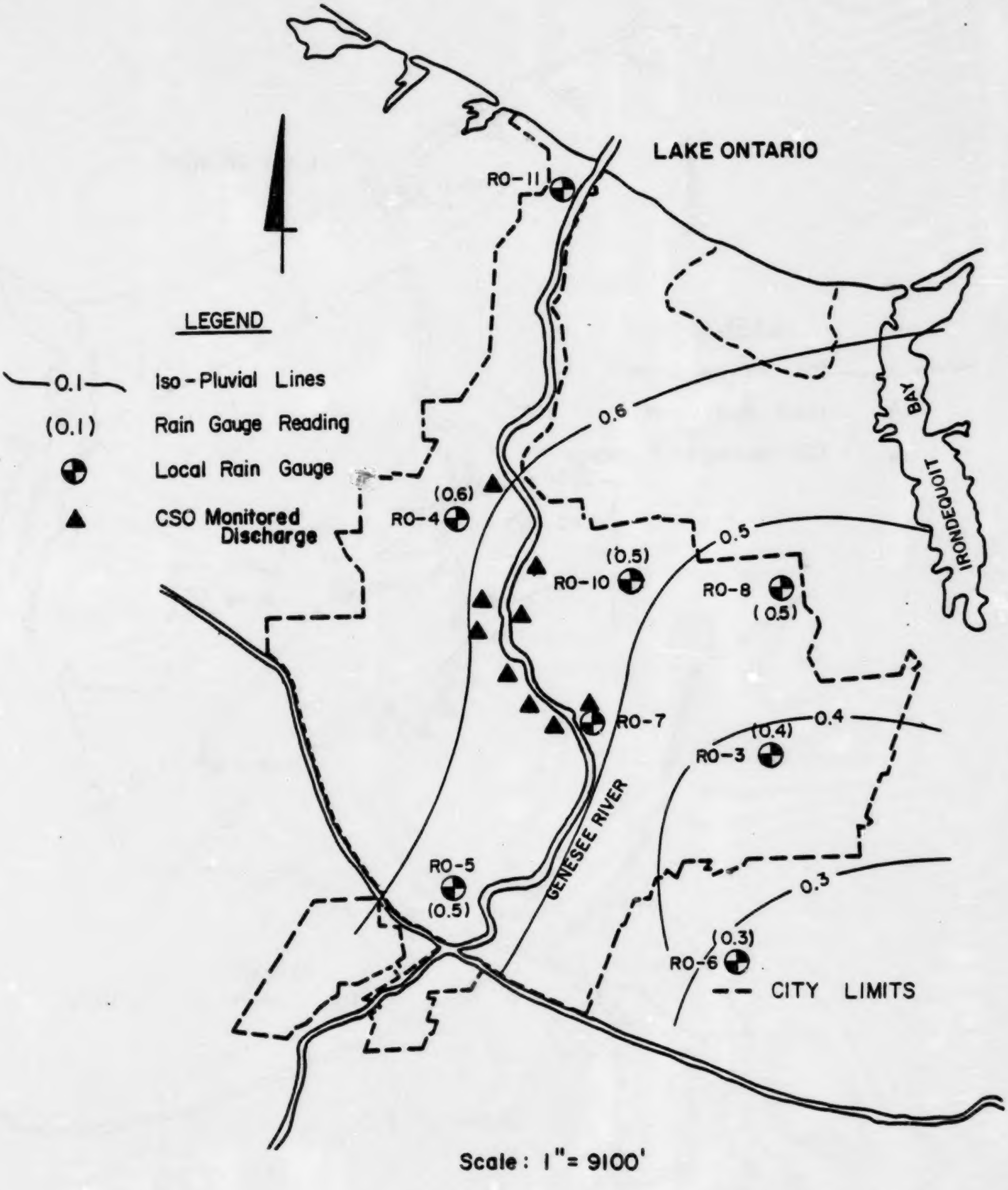

LAKE ONTARIO
LEGEND
0.1 Iso-Pluvial Lines
(0.1) Rain Gauge Reading
Local Rain Gauge
CSO Monitored Discharge
RO-11
RO-4
(0.6)
RO-10
(0.5)
RO-8
(0.5)
RO-7
RO-3
(0.4)
RO-5
(0.5)
RO-6
(0.3)
0.6
0.5
0.4
0.3
GENESEE RIVER
IRONDEQUOIT BAY
CITY LIMITS
Scale: 1" = 9100'

Figure 21. Iso-pluvial lines for 22 Jul 80 storm.

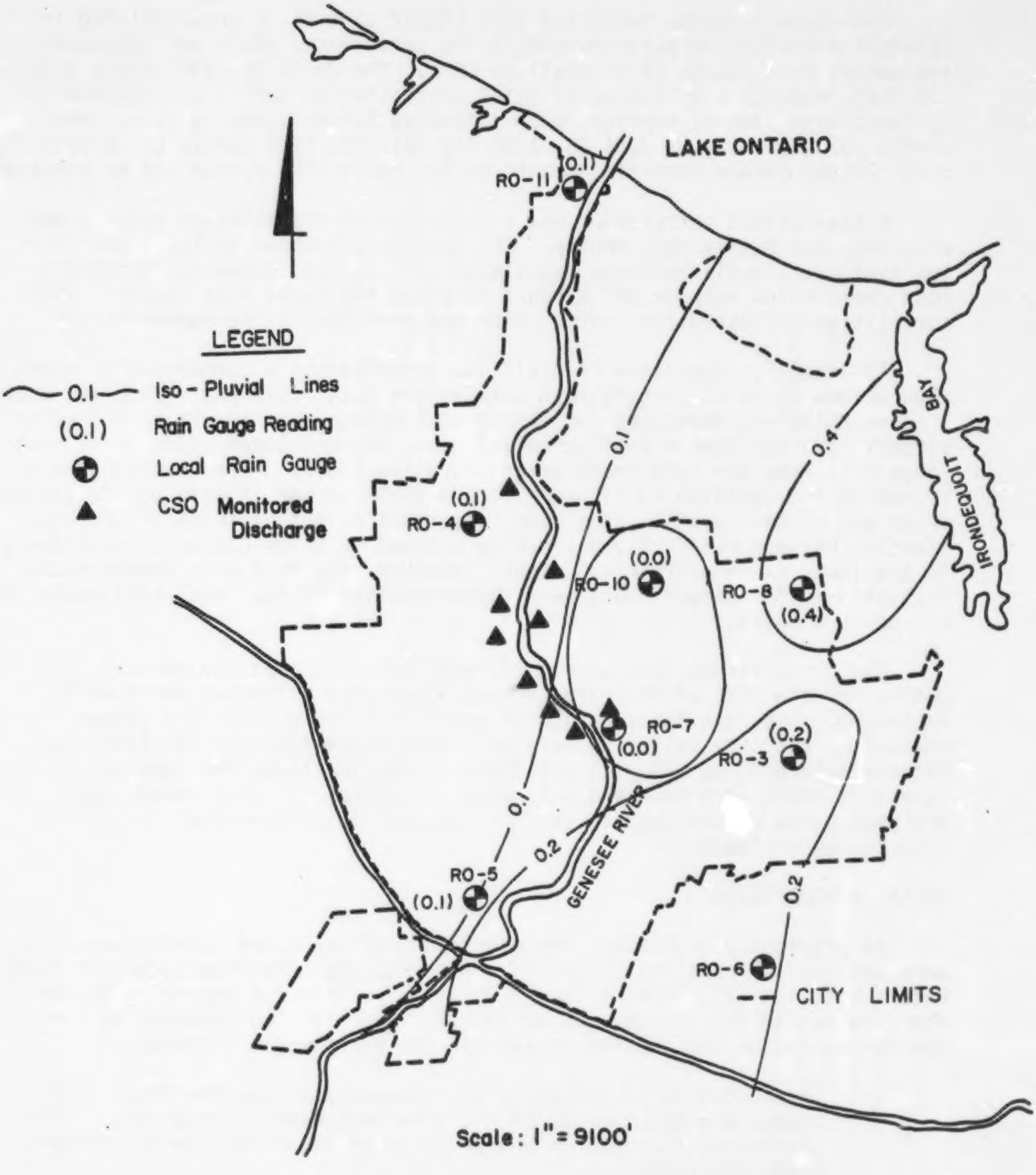

LAKE ONTARIO
LEGEND
0.1 Iso - Pluvial Lines
(0.1) Rain Gauge Reading
Local Rain Gauge
CSO Monitored Discharge
RO-11
(0.1)
RO-4
(0.1)
RO-10
(0.0)
RO-8
(0.4)
RO-7
(0.0)
RO-3
(0.2)
RO-5
(0.1)
RO-6
0.1
0.1
0.2
0.2
0.4
GENESEE RIVER
IRONDEQUOIT BAY
CITY LIMITS
Scale: 1" = 9100'

Figure 22. Iso-pluvial lines for 26 Jun 80 storm.

The above examples indicated that significant differences existed in rainfall distribution patterns within the urban area, which was addressed in evaluating the impacts of rainfall on CSOs. The March 21, 1980 storm indicated that reasonable estimates of total precipitation had to be obtained for cyclonic type events; however, such estimates for fast-moving, transient events such as those of June 26, 1980 and July 22, 1980 had to be evaluated more closely before specific impacts on the collection system can be assessed.

A statistical correlation was also conducted for selected storm events occurring between certain months. The results are shown in Table 16. For the typical rainfall patterns associated with spring, there was generally good correlation between the airport data and the local rain gauges. This correlation decreased for storms which occurred during the summer months.

To establish long-term rainfall-CSO correlations a comparison of overflow volume based on airport rain data versus local rain gauge data was made. A correlation was developed for the Central Avenue overflow (Site 36) using airport data and then a similar relationship was developed using local rain gauge data from the Fire Headquarters location (RO-7). The resulting relationships are depicted on Figure 23. The small number of data points at the upper end of the rainfall axis made assessment of the validity of the relationship between rainfall and overflow subject to a low degree of confidence. In the lower range of rainfall events, however, the data were somewhat consistent, and the airport data were representative of the local rain gauge on a long-term basis.

The correlations that were performed indicated that the rainfall distribution for the City of Rochester varies widely for different storm events. Because of this, the installation of eight local recording rain gauges throughout the city helped to more accurately determine the relationships between rainfall and CSO at the different overflow locations than precipitation data taken only from the U.S. Weather Bureau. It also showed that analyses based on the assumption that rainfall is uniform over the entire city can be misleading.

OVERFLOW MONITORING

As previously indicated, the overflow monitoring and sampling systems were originally installed under the former R&D grant for the Rochester Pure Waters District (1). During the BMP program, continuous upgrading of the overflow monitoring systems was conducted. Specific improvements to the monitoring system implemented as part of the BMP program included:

- Incorporation of telemetry instrumentation into the local rain gauges and the removal of the site recording strip charts. This eliminated the need to change paper on the strip chart recorders every two weeks.

- Replacement of all Badger Meter, Inc. ultrasonic head and velocity systems with portable ultrasonic head systems manufactured by Manning Corporation. With these units, more accurate and reliable

TABLE 16. CORRELATION OF LOCAL RAIN GAUGE DATA WITH NATIONAL WEATHER SERVICE DATA AT MONROE COUNTY AIRPORT

Rain Gauge	Distance From NWS Gauge at Airport	Number of Data Points	Coefficient of Correlation
	Data for Storms March - August		
East High School		34	0.55
Marshall H.S.		8	0.93
School #44		27	0.77
Brighton-Middle		25	0.53
Fire Headquarters		18	0.72
Norton Screenhouse		16	0.42
Franklin H.S.		17	0.61
Charlotte P.S.		27	0.59
	Data for Storms March - May		
East High School		15	0.86
Marshall H.S.		2	1.00
School #44		13	0.90
Brighton-Middle		11	0.76
Fire Headquarters		13	0.82
Norton Screenhouse		10	0.86
Franklin H.S.		12	0.71
Charlotte P.S.		15	0.86
	Data for Storms June - August		
East High School		19	0.54
Marshall H.S.		6	0.94
School #44		14	0.80
Brighton-Middle		14	0.50
Fire Headquarters		5	0.71
Norton Screenhouse		6	0.07
Franklin H.S.		5	0.51
Charlotte P.S.		12	0.34

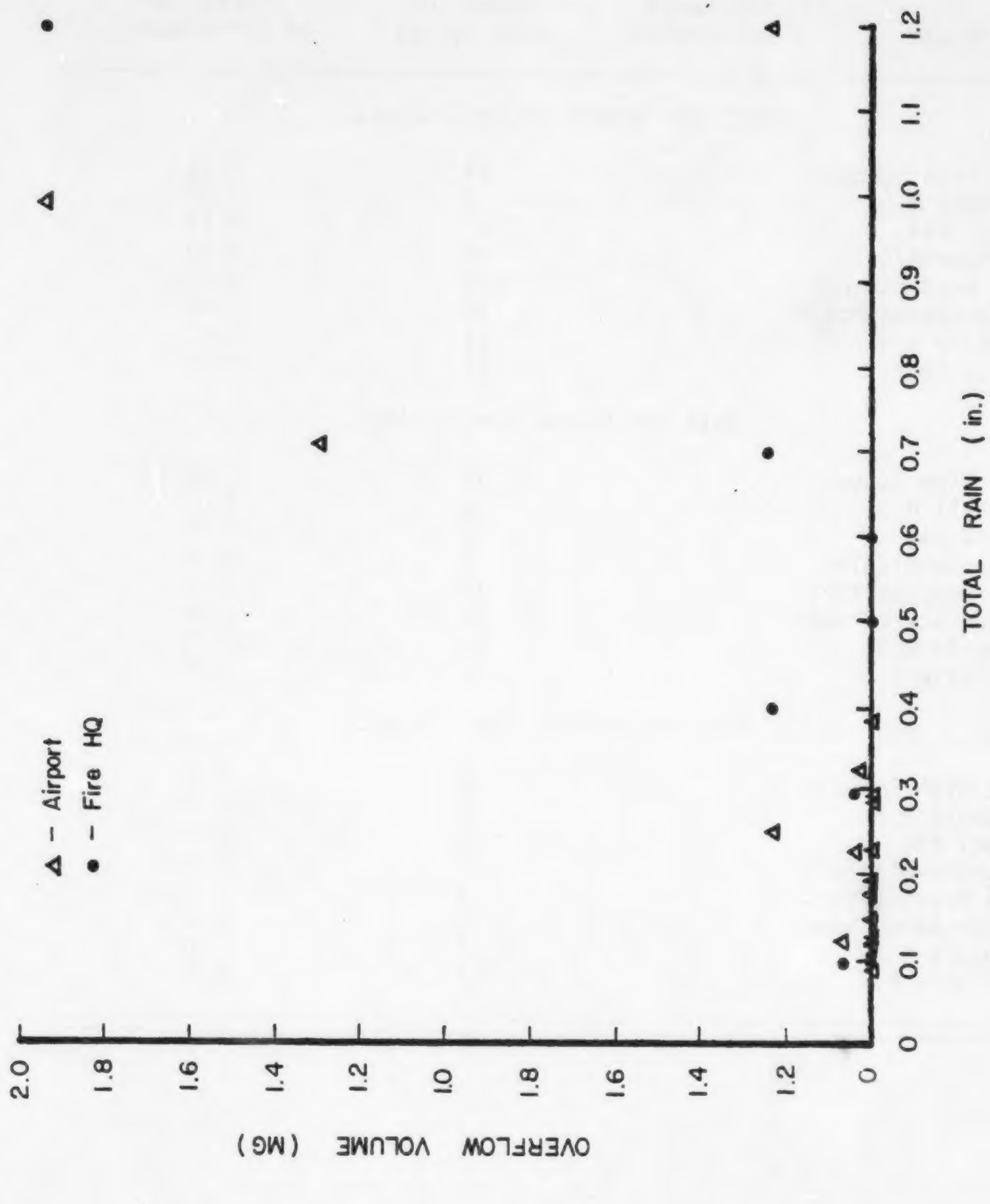


Figure 23. Airport rain data vs. local rain gauge data for site 36.

wastewater level measurements were obtained. Routine field maintenance was also simplified and expedited.

- Hardware and software modifications to the existing Digital Equipment Corporation PDP-8E computer located at the VanLare STP allowed for easier data acquisition and interpretation.
- Relocation of the level sensor to better monitor the anticipated flow conditions in the trunk sewers.
- Refinements to the characteristic flow equation for each overflow site which allowed for more accurate flowrates.
- Formation of a detailed and demanding routine field maintenance program. Because of this, many of the types of problems normally associated with similar monitoring programs were either eliminated or substantially minimized.

A summary of the overflow data collected under the BMP program, Tables 17 and 18 present the 1979 and 1980 overflow monitored data, respectively. Shown in these tables are the rainfall characteristics and overflow volumes for most of the storm events which occurred during 1979 and 1980 through August. Although a thorough maintenance program was conducted, there were times when difficulties were experienced with the field monitoring systems. This is to be expected with any monitoring system located in a corrosive environment and subjected to wide variations in temperature and humidity. Tables 17 and 18 represent a portion of all of the recorded data collected during the two years of active monitoring. At all sites, continuous flow monitoring was conducted throughout the year. Most of the occasional problems were related to the telemetry systems. Having a local strip chart recorder at each overflow site, however, allowed for accurate overflow data collection despite occasional telemetry problems.

The data presented in these two tables illustrate the number of storms and the associated overflow volumes for selected storm events. A much more detailed evaluation of the results is presented in subsequent sections.

OVERFLOW QUALITY CONSIDERATIONS

The basic purposes of the BMP overflow sampling program was four-fold:

- To determine the extent and magnitude of the "first-flush" phenomena for each overflow site.
- To determine the high-impacting overflow locations relative to the total CSO pollutant loads.
- To determine whether the pollutional characteristics of each overflow site have changed significantly since the 1975 sampling program.

TABLE 17. SUMMARY OF RAINFALL AND COMBINED SEWER OVERFLOW VOLUMES FOR 1979

Date	1979 Rainfall Characteristics*					Overflow Volume, MG								
	Depth in.	Duration hrs	I_{60}** in./hr	I_{avg}+ in./hr	ADD# hrs	Site Number 7	10	11	21	22	27	31	36	Total
Jan 24-25	0.73	18	0.09	0.041	150	17.63	-	-	-	0.82	0	0.50	-	18.95
Feb 23	0.18	6	0.07	0.030	250	10.40	-	12.00	-	0.56	0	0.00	0.16	23.12
Mar 5	0.11	3	0.09	0.037	226	9.10	-	-	-	0.44	0	0.00	0.00	9.54
Mar 5	0.21	9	0.05	0.023	8	-	-	-	-	0.38	0	0.00	0.06	0.44
Mar 10	0.11	5	0.04	0.022	98	0.61	0	-	-	0.00	0	0.00	0	0.61
Mar 25	0.13	3	0.10	0.043	354	0.27	-	1.00	0.00	0.00	0	0.00	0	1.27
Mar 29-30	0.37	6	0.10	0.062	119	1.13	0.01	4.21	-	0.46	0.00	-	0.19	6.00
Mar 31	0.09	2	0.08	0.045	27	0	0	1.75	-	0	0.00	-	0	1.75
Apr 2	0.53	12	0.15	0.044	41	1.50	0.02	21.00	-	0.83	0.00	-	0.27	23.62
Apr 14	0.19	2	0.14	0.095	276	0.35	0.00	0.46	0.00	0.08	0.00	0.00	0.00	0.89
Apr 25	0.18	2	0.14	0.090	267	0.01	-	-	-	0.05	0.00	-	0	0.06
Apr 26-27	0.19	7	0.04	0.027	33	0	-	-	0	0	0.00	-	0	0.00
Apr 27	0.29	9	0.06	0.032	5	0.01	-	0.04	0	0	0.00	0.15	0	0.20
May 3	0.12	11	0.03	0.011	157	0	-	-	-	0	0.00	-	0	0.00
May 12	0.61	1	0.61	0.610	212	1.05	-	-	-	1.29	0.00	-	0.36	2.70
May 13	0.06	4	0.02	0.015	6	0	-	-	-	0	0.00	-	0	0.00
May 15	0.11	1	0.11	0.110	56	0.03	-	-	-	0	0.00	-	0	0.03
May 21	0.24	4	0.13	0.060	139	-	-	-	-	0.03	0.00	-	0.12	0.15
May 24	0.63	6	0.16	0.105	66	-	-	-	-	2.15	0.00	-	0.55	2.70
May 25	0.26	4	0.14	0.065	21	-	-	-	-	2.43	0.00	-	0.62	3.05
May 26	0.39	6	0.13	0.065	18	-	-	-	-	1.64	0.88	-	0.51	3.03
May 27	0.07	3	0.04	0.023	21	-	-	-	-	0	0	-	0	0.00
May 28	0.11	2	0.07	0.055	20	-	-	-	-	0	0	-	0	0.00
May 29	0.44	5	0.14	0.088	23	-	-	-	-	0.13	0	-	0.08	0.21
Jun 5	0.10	4	0.06	0.025	106	-	-	-	-	0	0	-	0	0.00
Jun 7	0.34	2	0.18	0.170	38	2.95	-	-	-	1.57	0.64	-	1.26	6.42
Jun 8	0.07	1	0.07	0.070	15	0	-	1.67	-	0	0	-	0	1.67
Jun 10	0.14	1	0.14	0.140	52	0	-	-	-	0	0	-	0	0.00
Jun 10-11	0.31	4	0.17	0.078	9	0.33	-	-	-	0.39	0	-	0	0.72
Jun 22	0.15	4	0.05	0.038	273	1.39	0.04	2.72	1.13	0.44	0.05	0.94	0.17	6.88
Jun 28	0.35	9	0.13	0.039	131	0.23	0	0.93	0.29	0.56	0	0.28	0.26	2.55
Jun 29	0.12	3	0.06	0.040	26	0	0	0.00	0.00	0.00	0	0.01	0.00	0.01
Jun 30	0.23	3	0.09	0.077	22	1.00	0	1.57	1.42	1.39	0	0.50	0.57	6.45
Jul 10	0.26	4	0.16	0.065	224	0	0	-	0.00	0.01	0	0.03	0.00	0.04
Jul 11	0.12	1	0.12	0.120	19	0.52	0	-	0.00	0.00	0	0.01	0.00	0.53
Jul 11	0.13	1	0.13	0.130	4	-	0	-	0.00	0.16	0	0.01	0.01	0.18
Jul 14	0.41	3	0.21	0.137	47	0	0	-	0.07	0.76	0	0.00	0.00	0.83
Jul 15	0.68	2	0.58	0.340	18	2.95	0.58	-	3.94	4.69	0.06	1.96	0.20	14.38
Jul 16	0.09	2	0.06	0.045	17	0.02	0	-	0.00	0.00	0	0.20	0.00	0.22
Jul 23	0.23	1	0.23	0.230	174	0.00	0	-	0.00	0.00	0	0.00	0.00	0.00
Jul 24	0.12	1	0.12	0.120	20	3.10	1.44	1.46	0.09	0.00	0.17	4.50	0.07	10.83
Jul 26	0.27	6	0.09	0.045	32	0.51	-	3.01	0.78	0.53	0	0.37	0.59	5.79
Jul 31	0.70	4	0.58	0.175	126	0.85	-	2.25	2.08	4.56	0.04	5.12	1.84	16.74

(continued)

TABLE 17 (continued)

Date	1979 Rainfall Characteristics*					Overflow Volume, MG								
						Site Number								
	Depth in.	Duration hrs	I_{60}** in./hr	I_{avg}+ in./hr	ADD≠ hrs	7	10	11	21	22	27	31	36	Total
Aug 4	0.30	1	0.30	0.300	90	0.00	0	-	0.00	0.03	0	0.00	0.00	0.03
Aug 10	0.35	5	0.21	0.070	133	0.10	3.02	10.50	5.29	8.50	0.25	3.94	3.06	34.66
Aug 14	0.19	4	0.12	0.048	84	0.88	0.89	3.80	0.94	1.11	-	0.60	0.39	8.61
Aug 26-27	0.63	11	0.16	0.057	51	0	9.64	5.04	2.38	3.70	-	0.87	0.49	22.12
Aug 29-30	0.09	4	0.03	0.025	63	0	-	0.00	-	0.00	-	0.00	0.00	0.00
Sep 2	0.57	4	0.31	0.143	84	0	-	1.41	-	4.81	-	0.21	-	6.43
Sep 6	0.50	7	0.14	0.071	79	1.03	-	7.28	-	4.47	-	0.35	0.64	13.77
Sep 10	0.33	5	0.12	0.066	101	0	-	3.66	-	2.94	-	0.03	0.75	7.38
Sep 13-14	3.54	16	0.49	0.221	74	18.65	39.32	31.33	15.38	34.39	-	0.59	-	139.66
Sep 18	0.20	1	0.20	0.200	100	0	-	1.70	-	5.38	-	0.17	-	7.25
Sep 28	0.15	8	0.04	0.019	228	0	-	-	0.00	-	-	0.00	-	0.00
Oct 3	0.18	3	0.12	0.060	106	0	-	0.11	0.00	-	-	0.42	-	0.53
Oct 5	0.84	17	0.10	0.050	44	0.47	-	3.87	0.39	-	-	8.86	-	13.59
Oct 8-9	0.29	8	0.07	0.035	44	0.10	-	0.79	0.00	-	-	7.41	-	8.30
Oct 20	0.30	4	0.11	0.075	59	0	0.50	0.80	0.01	-	0	1.37	0.11	2.79
Oct 23	0.27	6	0.07	0.045	76	0	0.70	0.86	0.04	0.14	0	1.96	0.08	3.78
Oct 27-28	0.22	3	0.08	0.073	100	-	0.00	0.25	0.00	0.00	0	0.00	0.01	0.26
Nov. 7	0.23	7	0.06	0.033	240	-	0.00	0.41	0.00	0.00	0	1.80	0.00	2.21
Nov 9-10	0.29	13	0.04	0.022	54	-	0.00	0.78	0.00	0.00	-	5.60	0.00	6.38
Nov 24	0.14	4	0.07	0.035	18	0	0.00	0.00	0.00	0.00	0	0.98	0.00	0.98
Nov 26	0.38	7	0.12	0.054	18	1.10	0.00	2.97	1.06	0.78	0.16	18.78	0.61	25.46
Nov 28	0.06	1	0.06	0.060	37	0	0.00	0.00	0.00	0.00	0	0.22	0.00	0.22
Dec 6	0.10	2	0.05	0.050	205	0	-	-	0.00	0.00	0	0.49	0.00	0.49
Dec 23	0.16	4	0.08	0.040	300±	-	-	-	0.00	0.00	0	1.31	0.00	1.31
Dec 24-26	1.70	39	0.14	0.044	24	7.90	-	49.35	13.40	19.15	-	69.19	5.47	164.46

* US Weather Bureau data measured at Monroe County Airport.

** I_{60} = Maximum hourly intensity (with respect to clock hour) during storm event.

+ I_{ave} = Total rainfall depth divided by storm duration

≠ ADD = Antecedent dry period defined as number of hours separating identified storm events.

++ Total volume indeterminate due to excedence of flow meter measurement range.

TABLE 18. SUMMARY OF RAINFALL AND COMBINED SEWER OVERFLOW VOLUMES FOR 1980

	1980 Rainfall Characteristics*					Overflow Volume, MG++								
						Site Number								
Date	Depth in.	Duration hrs	I_{60}** in./hr	I_{avg}+ in./hr	ADP≠ hrs	7	10	11	21	22	27	31	36	Total
Jan 11	0.38	11	0.09	0.035	120	0.02	-	-	0.00	0.00	-	4.66	0.00	4.68
Jan 17	0.06	2	0.05	0.030	48	0.00	-	0.09	0.00	0.00	0.00	0.00	0.00	0.09
Mar 10	0.38	6	0.24	0.063	64	1.08	-	0.00	2.56	9.00	2.74	-	1.03	16.41
Mar 21-22	1.20	18	0.11	0.067	70	-	-	9.60	0.00	-	-	-	0.25	9.85
Mar 24	0.18	6	0.06	0.030	46	-	2.41	1.77	0.00	-	-	-	0.00	4.18
Mar 29	0.23	9	0.05	0.026	101	0.00	2.00	1.57	0.00	0.00	0.00	0.00	0.00	3.57
Mar 31	0.29	12	0.05	0.024	25	0.00	1.96	1.15	0.00	0.00	0.00	0.00	0.00	3.11
Apr 4	0.25	5	0.10	0.050	89	0.51	0.17	0.00	0.41	0.04	0.15	0.00	0.23	1.51
Apr 8-9	0.12	4	0.03	0.030	110	0.00	0.00	0.19	0.00	0.00	0.00	0.00	0.00	0.19
Apr 9	0.13	1	0.13	0.130	16	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.07	0.12
Apr 12	0.18	9	0.05	0.020	58	0.00	0.15	0.33	0.00	0.00	0.00	0.00	0.00	0.48
Apr 14-15	0.23	4	0.07	0.058	38	0.13	0.54	2.60	0.18	0.00	0.00	0.20	0.04	3.69
Apr 24	0.09	1	0.09	0.090	236	0.00	0.02	0.14	0.00	0.00	0.00	0.00	0.00	0.16
Apr 27	0.13	2	0.08	0.065	69	0.00	0.19	0.41	0.00	0.00	0.00	0.00	0.00	0.60
Apr 28	0.99	11	0.21	0.090	8	3.84	1.92	21.71	2.70	7.93	5.41	2.27	1.95	47.73
May 13	0.33	9	0.08	0.034	347	-	1.12	0.45	0.01	0.00	0.00	0.09	0.03	1.70
May 14	0.06	4	0.03	0.015	15	-	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
May 17-18	0.48	9	0.12	0.053	82	-	0.68	4.96	0.45	0.69	0.41	0.34	0.55	8.08
May 30	0.09	3	0.07	0.030	297	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
May 31	0.47	2	0.25	0.235	20	0.00	0.08	7.79	1.35	1.10	1.39	0.61	1.19	13.51
Jun 1	0.40	4	0.32	0.100	28	1.90	0.94	8.62	1.64	0.98	1.81	1.14	1.29	18.32
Jun 3	0.10	3	0.05	0.033	27	0.00	0.25	0.08	0.00	0.00	0.00	0.013	0.00	0.34
Jun 6-7	2.12	4	1.97	0.530	88	> 5.28	> 5.99	6.50	> 2.64	> 5.46	3.28	1.35	3.45	> 33.95
Jun 7-8	2.17	4	1.05	0.543	24	5.96	2.08	> 20.14	0.38	> 4.94	3.26	1.70	4.71	> 43.17
Jun 9	0.08	1	0.08	0.080	32	0.69	-	7.11	0.00	0.00	1.25	0.68	0.58	10.31
Jun 15	0.15	5	0.07	0.030	18	0.03	0.60	-	0.00	0.00	0.00	0.03	0.00	0.66
Jun 19-20	0.71	9	0.24	0.080	122	0.27	5.29	2.43	0.00	0.00	2.05	1.21	1.31	12.57
Jun 26	0.28	3	0.20	0.093	155	0.00	0.47	0.40	0.00	0.00	0.00	0.00	0.00	0.87
Jun 28	0.46	3	0.25	0.096	38	-	1.14	5.09	0.00	0.01	0.00	0.81	0.90	7.95
Jul 2	0.19	2	0.16	0.095	86	-	2.14	0.11	0.00	0.00	0.00	0.17	0.05	2.47
Jul 8	0.09	2	0.07	0.044	144	-	1.22	0.00	0.00	0.00	0.00	0.00	0.00	1.22
Jul 22	0.58	2	0.45	0.290	329	0.72	0.14	1.06	0.12	0.44	1.24	0.00	0.82	4.54
Jul 22	0.45	8	0.20	0.056	4	0.77	1.74	0.00	0.00	0.00	0.12	0.00	0.55	3.18
Jul 27	0.16	3	0.11	0.053	121	-	0.50	0.38	0.00	0.00	0.00	0.00	0.00	0.88
Jul 28-29	0.10	2	0.09	0.050	25	0.10	0.22	0.00	0.00	0.00	0.00	0.00	0.00	0.32
Aug 2	0.13	4	0.06	0.033	84	0.30	0.23	0.00	-	0.00	2.14	0.00	0.00	2.67
Aug 3	0.60	8	0.17	0.075	4	1.97	0.68	> 10.00	-	1.05	2.47	> 0.60	1.44	> 18.21
Aug 4	0.72	1	0.72	0.720	51	> 4.24	-	> 8.75	0.002	> 1.61	1.96	> 0.68	2.72	> 19.96
Aug 5-6	0.60	6	0.26	0.100	4	7.76	-	> 15.94	0.062	0.23	0.00	> 1.46	1.99	> 27.44
Aug 14-15	1.24	2	0.71	0.620	198	0.47	0.06	0.00	0.00	0.28	0.00	> 0.39	0.01	> 1.21

* U.S. Weather Bureau data as measured at Monroe County Airport

** I_{60} = Maximum hourly intensity (with respect to clock hour) during storm event

+ I_{avg} = Total rainfall depth divided by storm duration

≠ ADP = Antecedent dry period defined as number of hours separating identified storm events

++ Total volume at some sites (indicated by >) indeterminate due to excedence of flowmeter measurement range

- To determine the pollutional characteristics of each overflow site after implementation of the proposed regulator/weir modifications.

Analytical data from the 1979-1980 BMP monitoring program were statistically analyzed to provide geometric mean concentrations of selected pollutant parameters within pre-determined time intervals after the start of an overflow event. The time interval selected for analysis was 30-min which yielded three intervals for the first 90-min of overflow and a final interval extending from 90-min to the end of the overflow event.

A 30-min interval was selected on the basis of the 15-min sampling frequency used for the monitoring program, and on an assumption that large concentration changes could be identified by such an interval. The statistical analysis provided mean concentration profiles for BOD, TSS, TKN and TIP. The first-flush mean concentration data are presented in Table 19.

From Table 19, a "first-flush", in which a disproportionately high pollutional load is carried in the first portion of the overflow, was generally observed for each CSO location. These higher concentrations during the initial portion of storms has been attributed to the scour and washoff of materials from street surfaces and to the resuspension of materials and debris which may have settled along the sewer invert during periods of dry-weather. Once dirty street surfaces have been "cleaned" by the impact of raindrops and removed by runoff, and once the settled materials along pipe inverts are resuspended and carried downstream, the pollutant concentrations can be largely attributed to normal surface erosion and the sanitary waste characteristics.

The implications of controlling the "first-flush" portion of CSO's are clear. If the first 60-min of a CSO could be contained within the sewer system, the total pollutant load discharged to the receiving water would be substantially reduced.

A comparison of selected overflow quality data collected under the BMP program to data collected under the previous R&D program is shown in Table 20. Inspection of Table 20 indicates that the quality of Rochester's CSO's had changed little since the 1975 sampling program. Arithmetic mean values for BOD (computed on a system-average basis) were 158 mg/l in 1975 and 183 mg/l during 1979 to 1980, whereas TSS exhibited an increase from 376 mg/l in 1975 to 509 mg/l in 1979 - 1980. TKN increased approximately 11 percent in that period. However, in examining the data for individual overflow sites, it was observed that Seth Green (Site 27) exhibited a marked reduction in BOD and TSS. As described in later sections of this report, Seth Green is the overflow relief point for the East Side Trunk Sewer which drains a large portion of the City's combined sewer system. The ESTS is in need of rehabilitative measures due to the accumulation of debris and silt on the invert (crown failure and sewer backup problems). Deposition of the settleable fractions of BOD and TSS as a result of the sewer backup conditions could be a cause of the reduction in overflow characteristics, although serious debris problems had been identified prior to both overflow monitoring programs (6).

TABLE 19. FIRST-FLUSH CONCENTRATIONS BY OVERFLOW SITE

		Geometric Mean Concentration, mg/l							
		BOD_5				TSS			
Drainage Area		30*	60	90	> 90	30	60	90	> 90
7	Maplewood	148	67	36	32	569	399	210	157
10	Lexington	127	58	55	34	324	199	238	140
11	WSTS	158	130	69	-	1043	1200	710	-
21	Mill & Factory	239	111	30	-	760	432	550	-
22	Front	118	112	79	58	553	473	401	203
27	Seth Green	76	88	70	67	454	438	364	365
31	Carthage	533	397	410	429	1054	977	829	512
36	Central	61	57	45	43	221	223	161	130
		TKN				TIP			
		30	60	90	> 90	30	60	90	> 90
7	Maplewood	5.36	2.55	2.24	1.20	1.55	1.34	0.67	0.45
10	Lexington	3.71	2.70	1.32	1.09	1.17	0.68	0.61	0.47
11	WSTS	6.94	5.51	3.14	-	2.52	2.81	1.62	-
21	Mill & Factory	5.15	4.09	2.60	-	1.33	1.12	0.49	-
22	Front	7.21	6.67	7.59	7.46	1.72	1.38	1.40	1.07
27	Seth Green	2.49	2.51	2.80	3.13	1.30	1.18	1.11	1.22
31	Carthage	9.59	3.47	7.03	5.17	3.30	1.84	1.65	1.29
36	Central	1.33	1.97	2.76	5.48	0.77	0.97	0.82	0.68

* Time interval ending at given minute.

TABLE 20. COMPARISON OF 1975 OVERFLOW QUALITY DATA WITH 1979-1980 DATA

Drainage Area	R&D Program			BMP Program		
	No. Data Points	Arithmetic Mean	Geometric Mean	No. Data Points	Arithmetic Mean	Geometric Mean
	BOD, mg/l					
7 Maplewood	126	134 ± 28	79	116	92 ± 58	50
10 Lexington	93	69 ± 18	47	89	94 ± 18	59
11 WSTS	13	130 ± 32	118	6	120 ± 42	113
21 Mill & Factory	21	97 ± 33	74	28	163 ± 82	82
22 Front	-	-	-	19	100 ± 30	83
27 Seth Green	15	136 ± 94	75	63	77 ± 15	57
31 Carthage	282	478 ± 49	308	128	767 ± 99	482
36 Central	23	61 ± 22	49	23	51 ± 11	46
Systemwide Mean Values						
(incl. Site 31)		158	140		183	102
(excl. Site 31)		105	65		100	57
	TSS, mg/l					
7 Maplewood	135	449 ± 80	247	116	361 ± 89	190
10 Lexington	103	129 ± 15	111	89	261 ± 86	165
11 WSTS	22	220 ± 46	200	6	1000 ± 301	961
21 Mill & Factory	21	474 ± 280	306	28	443 ± 144	318
22 Front	-	-	-	19	486 ± 231	298
27 Seth Green	15	511 ± 370	269	63	377 ± 57	293
31 Carthage	286	591 ± 65	438	128	960 ± 209	637
36 Central	23	258 ± 71	212	23	184 ± 32	172
Systemwide Mean Values						
(incl. Site 31)		376	280		509	290
(excl. Site 31)		340	201		445	216

(continued)

TABLE 20 (continued)

Drainage Area	R&D Program No. Data Points	R&D Program Arithmetic Mean	R&D Program Geometric Mean	BMP Program No. Data Points	BMP Program Arithmetic Mean	BMP Program Geometric Mean
			TIP, mg/l			
7 Maplewood	184	1.21 ± 0.48	-	116	1.21 ± 0.56	0.62
10 Lexington	157	0.26 ± 0.04	-	89	0.95 ± 0.26	0.65
11 WSTS	24	1.01 ± 0.24	0.35	6	2.33 ± 0.58	2.25
21 Mill & Factory	22	0.21 ± 0.08	0.16	28	1.01 ± 0.25	0.86
22 Front	-	-	-	19	1.43 ± 0.34	1.28
27 Seth Green	14	0.41 ± 0.28	0.25	63	1.22 ± 0.31	0.86
31 Carthage	386	1.76 ± 0.61	-	127	2.32 ± 0.30	1.76
36 Central	34	0.43 ± 0.13	0.30	23	0.78 ± 0.12	0.28
Systemwide Mean Values						
(incl. Site 31)		0.76	0.33		1.41	0.88
(excl. Site 31)		0.59	0.33		1.28	0.68
			TKN, mg/l			
7 Maplewood	181	4.41 ± 0.60	-	116	3.91 ± 0.85	2.27
10 Lexington	157	3.70 ± 0.65	-	88	5.71 ± 1.42	2.38
11 WSTS	24	11.07 ± 4.10	6.92	6	5.26 ± 1.89	4.93
21 Mill & Factory	22	1.86 ± 0.81	1.33	28	3.98 ± 1.77	2.41
22 Front	-	-	-	19	7.67 ± 0.59	7.57
27 Seth Green	14	2.60 ± 0.57	2.40	63	3.04 ± 0.48	2.54
31 Carthage	377	7.63 ± 0.80	4.98	127	12.35 ± 2.33	6.69
36 Central	34	3.96 ± 0.73	3.38	23	3.54 ± 0.89	2.58
Systemwide Mean Values						
(incl. Site 31)		5.03	4.42		5.61	3.33
(excl. Site 31)		4.60	3.10		4.65	2.56

(continued)

TABLE 20 (continued)

Drainage Area	R & D Program No. Data Points	R & D Program Geometric Mean	BMP Program No. Data Points	BMP Program Geometric Mean
	Fecal Coliform (MPN/100 ml) x 10^6			
7 Maplewood	139	0.35	116	0.24
10 Lexington	99	0.15	89	0.16
11 WSTS	15	0.14	6	0.21
21 Mill & Factory	21	0.25	28	0.31
22 Front	-	-	18	1.40
27 Seth Green	15	0.77	63	0.35
31 Carthage	255	0.54	128	0.37
36 Central	23	0.12	23	0.16
Systemwide Mean Values				
(incl. Site 31)		0.35		0.28
(excl. Site 31)		0.24		0.25

A second major observation is discernible from Table 20, which is associated with the Carthage overflow location (Site 31). Increases in BOD, TSS, and TKN of more than 60 percent were observed from 1975 to 1979-1980. One large industry discharges a heavy organic loading into the Carthage Avenue trunk sewer. In the last two to three years a continuing urban renewal project has resulted in an estimated 30 to 40 percent removal of tributary stormwater drainage to the Carthage Avenue trunk sewer. The loss of diluting stormwater was suspected to be the major reason why the overflow characteristics at Carthage have risen so significantly.

If the overflow characteristics of the Carthage site are not included in the computation of system-wide pollutant concentrations, the average BOD would be reduced to 105 mg/l for 1975 data and 100 mg/l for 1979-1980 data; TSS would be reduced to 340 mg/l for 1975 data and 445 mg/l for 1979-1980; and similarly, TKN would be reduced to 4.60 mg/l and 4.65 mg/l for 1975 and 1979-1980 data, respectively.

Examination of fecal coliform data indicated that little change has occurred in the concentrations of bacteria discharged via CSO's since 1975. In general, a dramatic change in drainage area characteristics (either surface runoff or sewage) would be required to affect a significant change in bacterial concentrations, since the volumes required to dilute bacteria to significantly lower levels would be substantial.

Comparison of the geometric mean concentrations for the parameters discussed above, indicated that the overall pollutant characteristics of the combined sewer drainage area have changed little over the period from 1975 to 1979-1980, as summarized in Table 21.

TABLE 21. SYSTEMWIDE MEAN POLLUTANT CONCENTRATIONS FOR 1975 AND 1979-1980 BMP SAMPLING PERIOD

	Systemwide Mean Pollutant Concentrations			
	1975		1979-1980	
Pollutant	Arithmetic	Geometric	Arithmetic	Geometric
BOD, mg/l	105	65	100	57
TSS, mg/l	340	201	445	216
TKN, mg/l	4.60	3.10	4.65	2.56
TIP, mg/l	0.59	0.33	1.28	0.68
Fecal Coliform MPN/100 ml	-	240,000	-	250,000

One exception to the relatively good comparison of 1975 overflow quality data to 1979-1980 data was that of the means for TIP, wherein a twofold increase in TIP was observed. The cause for the increased level of phosphorus was not evident; however, it should be noted that the 1979-1980 TIP level approaches the 1.0 mg/l standard for point-source discharges into the Great Lakes drainage basins.

Several conclusions derived from analyzing the monitoring data are:

1. Drainage area 31 (Carthage) had mean pollutant concentrations ranging from 48 to 355 percent higher than the overall drainage basin mean concentrations, depending on the specific pollutant.

2. In general, Drainage areas 10 (Lexington) and 36 (Central) exhibited lower mean concentrations than did the remaining overflows.

3. Comparisons of individual drainage area pollutant characteristics with the overall drainage basin mean concentrations indicated that all sites exhibited mean concentrations within 50 percent of the drainage basin mean for BOD, TSS and TIP, and within 65% for TKN.

Although the above analysis of CSO quality characteristics can be utilized to determine the relative differences in quality between sites, the determination of a high impact ranking must be based on a mass loading contribution rather than on a pollutant concentration basis. To establish the pollutant loading relationships between overflow sites, a linear regression analysis was performed for each site relating overflow volume to total precipitation for a series of monitored storm events in 1980. The data utilized is presented in Table 22, with the equations developed for each of these sites presented in Table 23. When the regression equations were plotted as in Figure 24, it was noted that the total volume of overflow for specified rainfall events varied from site to site. Therefore, a ranking of the overflow sites based on pollutant loading characteristics must consider both quality and quantity. Using the average 1980 overflow quality characteristics presented in Table 20, the pollutant loadings from each site were projected for a storm of 1.0 in. of total precipitation and the results presented in Table 22. Based on these results, a ranking of high-impacting overflows was then developed for each of the quality parameters, as illustrated in Table 24. To assess the overall site ranking, the rankings of individual parameters were totaled, assuming the severity of impact of individual parameters was equivalent. If the severity of impact for particular parameters were considered non-uniform, a weighting factor could be assigned to the paramaters to allow for the greater severity of particular pollutants.

As exhibited in Table 24, the WSTS overflow ranked as the highest impacting overflow, followed by Front Street and Carthage. Maplewood, Lexington and Seth Green were of moderate impact (relative to the other sites), and Mill & Factory and Central were the least impacting of the overflow sites.

TABLE 22. PROJECTED POLLUTANT LOADINGS FOR A STORM OF 1.0 IN. OF TOTAL PRECIPITATION

Discharge Area	Total Discharge Volume, MG	Average Pollutant Concentrations, mg/l					Pollutant Loadings, lbs**				
		BOD	TSS	TKN	TIP	FC*	BOD$_5$	TSS	TKN	TIP	FC*
7 Maplewood	2.44	92	361	3.91	1.21	0.24×10^6	1870	7350	80	25	2.2×10^{13}
10 Lexington	2.25	94	261	5.71	0.95	0.16×10^6	1760	4900	107	18	1.4×10^{13}
11 WSTS	7.50	120	1000	5.26	2.33	0.21×10^6	7510	62550	329	146	6.0×10^{13}
21 Mill & Factory	1.03	163	443	3.98	1.01	0.31×10^6	1400	3800	34	9	1.2×10^{13}
22 Front	2.37	100	486	7.67	1.43	1.40×10^6	1980	9610	151	28	12.5×10^{13}
27 Seth Green	1.98	77	377	3.04	1.22	0.35×10^6	1270	6230	50	20	2.6×10^{13}
31 Carthage	1.01	767	960	12.35	2.32	0.37×10^6	6460	8090	104	20	1.4×10^{13}
36 Central	1.64	51	184	3.54	0.78	0.16×10^6	700	2520	48	11	1.0×10^{13}
Total	20.22						22950	105050	903	277	28.3×10^{13}

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* FC = Fecal Coliform expressed as MPN/100 ml (concentration) and MPN (loading)

** Figures in parentheses indicate percent of total loading.

TABLE 23. REGRESSION EQUATIONS BY OVERFLOW SITE CORRELATING OVERFLOW VOLUMES TO TOTAL RAINFALLS

Site	m	b	r	Equation
7	2.807	- 0.37	0.94	OF = 2.807 TR - 0.37
10	1.800	0.45	0.65	OF = 1.800 TR + 0.45
11	7.442	0.06	0.71	OF = 7.442 TR + 0.06
21	0.890	0.14	0.47	OF = 0.890 TR + 0.14
22	2.584	- 0.21	0.58	OF = 2.584 TR - 0.21
27	2.030	- 0.05	0.75	OF = 2.030 TR - 0.05
31	0.848	0.16	0.45	OF = 0.848 TR + 0.16
36	1.848	- 0.21	0.90	OF = 1.848 TR - 0.21
Σ	20.25	- 0.03		

Total System OF = 20.25 TR - 0.03 from summation of individual equations.

Total System OF = 18.05 TR + 0.31 from regression analysis of Table 18.

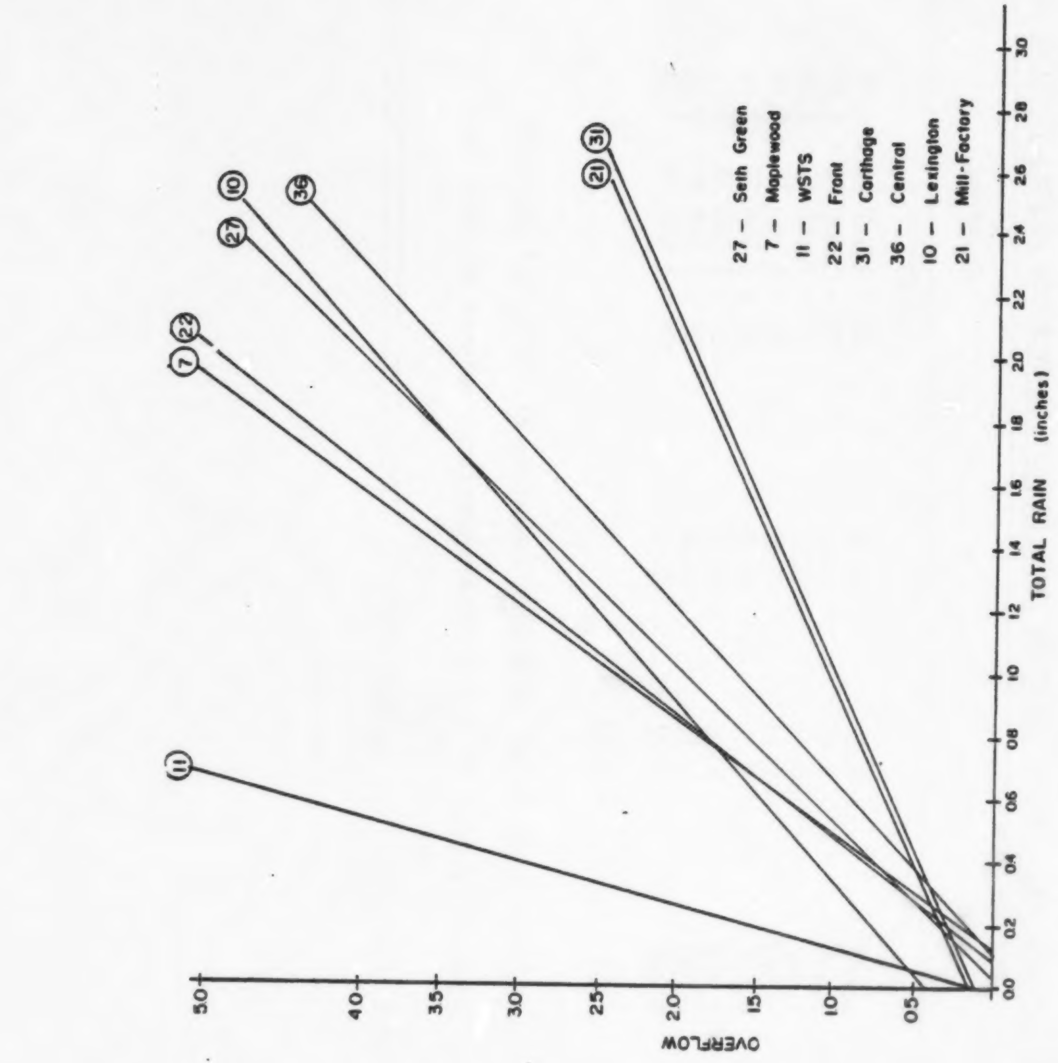


Figure 24. Rainfall - overflow regression equations by site.

TABLE 24. RANKING OF HIGH IMPACTING OVERFLOWS BASED ON POLLUTANT LOADINGS

Drainage Area	High Impact Ranking (1 = highest) BOD	TSS	TKN	TIP	FC	Ranking	Overall Rank
7 Maplewood	4	4	5	3	4	20	4
10 Lexington	5	6	3	6	2	22	5
11 WSTS	1	1	1	1	5	9	1
21 Mill & Factory	6	7	8	8	7	36	7
22 Front	3	2	2	2	1	10	2
27 Seth Green	7	5	6	5	3	26	6
31 Carthage	2	3	4	4	6	19	3
36 Central	8	8	7	7	8	38	8

SECTION 6

SOURCE CONTROL MANAGEMENT

CATCHBASIN/STREET SWEEPING EVALUATIONS

Background

At the heart of a BMP pollution abatement program are those source management practices which address the removal of contaminants where they accumulate before they are washed into the sewer collection system. A source control measure that has received considerable attention over the past several years is more effective catchbasin cleaning and street sweeping. If pollutants that accumulate on all land surfaces are somehow removed before entering a combined or storm sewer network, then they will not be discharged to receiving waters through CSOs or stormwater outlets.

Past research, notably that conducted for the USEPA by the American Public Works Association and the URS Research Company, has clearly revealed the water pollution potential of street surface contaminants (15,16,17,18). Although more research is needed to establish the quantitative effect of surface contaminants on receiving water quality, these previous projects presented strong evidence for a cause and effect relationship between accumulated surface pollutants and impaired receiving water quality.

As an indication of the importance of controlling surface contaminants, Table 25 presents observed runoff water quality concentrations as determined by a 1979 study conducted in San Jose, California (19).

Existing Maintenance Program

Although the cleaning of catchbasins is a function of the Monroe County Division of Pure Waters, street sweeping operations are conducted by the City of Rochester Department of Environmental Services (DES). Current street sweeping operations follow refuse collection in the City. Intensive cleaning efforts are made during both the spring and fall of the year. Normal summer operations have established frequencies consistent with the type of road use. On the average, residential areas are swept once every sixth working day, whereas, commercial/industrial areas are swept every day between the hours of 5 AM and 1 PM. The residential frequency has been established to avoid alternate parking problems. For both areas scheduled cleaning operations can be delayed several days because of adverse weather conditions.

TABLE 25. OBSERVED RUNOFF WATER QUALITY CONCENTRATIONS FOR SAN JOSE STUDY (19)

Parameter, Units*	Number of Analyses	Minimum	Maximum	Average
Common Parameters and Major Ions:				
pH	88	6.0	7.6	6.7
Oxidation Reduction Potential, mV	39	40	150	120
Temperature, ^{O}C	11	14	17	16
Calcium	5	2.8	19	13
Magnesium	5	1.4	6.2	4.0
Sodium	5	< 0.002	0.04	0.01
Potassium	5	1.5	3.5	2.7
Bicarbonate	5	< 1	150	54
Carbonate	5	< 0.001	0.005	0.019
Sulfate	5	6.3	27	18
Chloride	5	3.9	18	12
Solids:				
Total Solids	20	110	450	310
Total Dissolved Solids	20	22	376	150
Suspended Solids	20	15	845	240
Volatile Suspended Solids	10	5	200	38
Turbidity, NTU**	88	4.8	130	49
Specific Conductance, umhos/cm	88	20	660	160
Oxygen and Oxygen Demanding Parameters:				
Dissolved Oxygen	11	5.4	13	8.0
Biochemical Oxygen (5-day)	13	17	30	24
Chemical Oxygen	13	53	520	200
Nutrients:				
Kjeldahl Nitrogen	13	2	25	7
Nitrate	5	0.3	1.5	0.7
Orthophosphate	13	0.2	18	2.4
Total Organic Carbon	5	19	290	110
Heavy Metals:				
Lead	11	0.10	1.5	0.4
Zinc	11	0.06	0.55	0.18
Copper	11	0.01	0.09	0.03
Chromium	11	0.005	0.04	0.02
Cadmium	11	< 0.002	0.006	< 0.002
Mercury	11	< 0.0001	0.0006	< 0.0001

* mg/l unless otherwise noted

** Nephelometric turbidity units

The difference in sweeping efficiency depends to a large degree on the type of road surface, longitudinal slope, crown slope, and curb conditions. Sweeping efficiency also depends largely on the operator. The City of Rochester DES has reported that curbed streets have a relatively high degree of cleanability. Streets with concrete gutters are more difficult to clean than curbed streets, and streets with unpaved shoulders are the most difficult to clean effectively by sweeping.

The vehicle presently used the by the City of Rochester is the Elgin & Wayne mechanical sweeper. Estimated efficiencies of these sweepers, as reported by the DES, range between 60 and 70 percent (total solids) during the spring and fall months, while during the summer efficiency may average between 80 and 90 percent. In addition, sweeper maintenance is high due to the nature of the material on the streets. Fine particulates and dust, agitated from the roadway surface during normal cleaning operations, enters the bearings and other moving parts of the cleaning equipment and causes considerable mechanical wear.

The estimated capacity of the sweepers is about 3 cy. Material collected by the sweepers is taken to a transfer station for subsequent disposal at a sanitary landfill.

There are presently about 564 miles of streets being swept within the City of Rochester, covering an area of approximately 36 mi^2. Cost estimates for the present street sweeping/cleaning operations are about \$12/curb-mi. or approximately 1.1 mil \$/yr (1976 dollars). This cost includes manpower, vehicle purchase, maintenance, and depreciation, and transporting and disposal of collected solids.

A discussion on the existing sewer cleaning and maintenance program is presented because of its close association with catchbasin cleaning. This section represents a brief summary of the effectiveness of sewer flushing and routine sewer maintenance on the reduction of property calls (complaints). All of the work herein described is conducted by the Operations and Maintenance Division of the Monroe County Division of Pure Waters. The preventive maintenance program including sewer flushing was established by the Division in 1974 with the goal of decreasing the number of house complaints through more intensive sewer maintenance. The data as presented herein related to operations conducted during 1977 and 1978.

During 1977, Pure Waters reported a very significant (23%) decrease in the number of property calls within the Gates-Chili-Ogden and Rochester Districts. This was quite encouraging since the Division had increased their service areas, areas of responsibility, and number of people served. Their success in reducing the number of complaints appeared to be directly related to their highly organized preventive maintenance program.

As a brief overview, the Division of Pure Waters operates four hydro-flushing, vacuum cleaners, two hydro-flushers, three buckets, and one rodder. The operation involving the hydro-flusher also provides the televised sewer inspections. The hydro-flushing, vacuum cleaner unit is capable of storing 11 cy of solid debris and holding 1500 gal of water. At current prices a new hydro-flushing, vacuum cleaner costs approximately $100,000. A hydro-flusher unit only provides for sewer flushing and costs approximately $30,000. To satisfactorily perform the cleaning operations, each hydro-flushing, vacuum cleaner requires a two man crew, each bucket a four man crew, and each hydro-flusher only one man. Because of the material being collected and the heavy demands placed on the cleaning and flushing equipment, preventative maintenance on these units is essential.

The goal of the Division is to provide complete sewer maintenance and cleaning, including flushing for the entire City of Rochester sewer system once every five years. The primary emphasis of the maintenance operations is generally on reducing property calls (complaints) and relieving main sewer stoppages. The maintenance program began in 1974 and by the end of 1979 the entire City sewer system had been completely cleaned once. The normal work rate consists of one hydro-flushing, vacuum cleaner, and one bucket unit each operating within an area of approximately 1 mi. A hard working crew operating a hydro-flusing, vacuum cleaner can flush 1400 linear ft of sewer and clean eight catchbasins per day.

The effectiveness of the overall sewer flushing and maintenance program for the Rochester Pure Waters District can be seen from Table 26. Costs associated with conducting such a maintenance program are presented in Table 27.

TABLE 26. SUMMARY OF SEWER FLUSHING AND MAINTENANCE EFFECTIVENESS

Description	Year		
	1975	1976	1977
Total Property Calls (Complaints)	12,046	5,805	4,413
Footage of Main Sewer Cleaned	635,883	235,240	374,603
Main Sewer Stoppages	333	152	169
Catchbasins Cleaned	6,117	3,896	3,596
Footage of Sewers Televised	44,335	37,603	43,087

TABLE 27. COSTS ASSOCIATED WITH SEWER FLUSHING/MAINTENANCE PROGRAM

MAIN SEWER	
Total Length Hydrocleaned	57,725 LF
Total Hours Charged-Hydrocleaning	2,234
Unit Cost-Hydrocleaning	$ 0.27/LF*
Total Length Bucket - Rod	44,328 LF
Total Hours Charged-Bucket-Rod	7,709
Unit Cost-Bucket-Rod	$ 1.22/LF*
CATCHBASIN CLEANING	
Number Cleaned	1,501
Hours Charged	4,951
Unit Cost	$ 23.12/CB*

*Costs include manpower only. A further detailed cost breakdown is presented in the following subsection on street sweeping.

Demonstration Program Description

In an effort to quantify the effects of increased street sweeping and catchbasin cleaning on surface pollutants washed into the sewer system during rainfall, a simplified approach was taken as part of the source control management options investigated under the BMP study. Specifically, the purpose of the catchbasin/street sweeping study was three-fold:

- To quantify the mass pollutant loadings entering the sewer system during rainfall from catchbasins in areas of different land use.
- To determine the effectiveness of increased street sweeping on reducing these loadings.
- To compare the cost-effectiveness of increased system maintenance relative to other BMP and related pollution abatement options.

The catchbasin/street sweeping program was conducted during 1979 in cooperation with the City of Rochester Department of Environmental Services. The two most important land use areas investigated were residential and commercial. Because of the large volume of daily vehicular traffic for both of these areas, they represented the areas most responsible for runoff containing the highest pollutant loads.

The residential demonstration area was located on the west side of the City of Rochester and consisted of a two block area along Canton Street. The corresponding test and control areas for the commercial land use site were located along Humbolt Street on the east side of the City. Figures 25 through 27 show the general location of the demonstration sites and schematics of the test and control areas for each land use. Figures 28 and 29 are photographs of the actual test areas for both land uses.

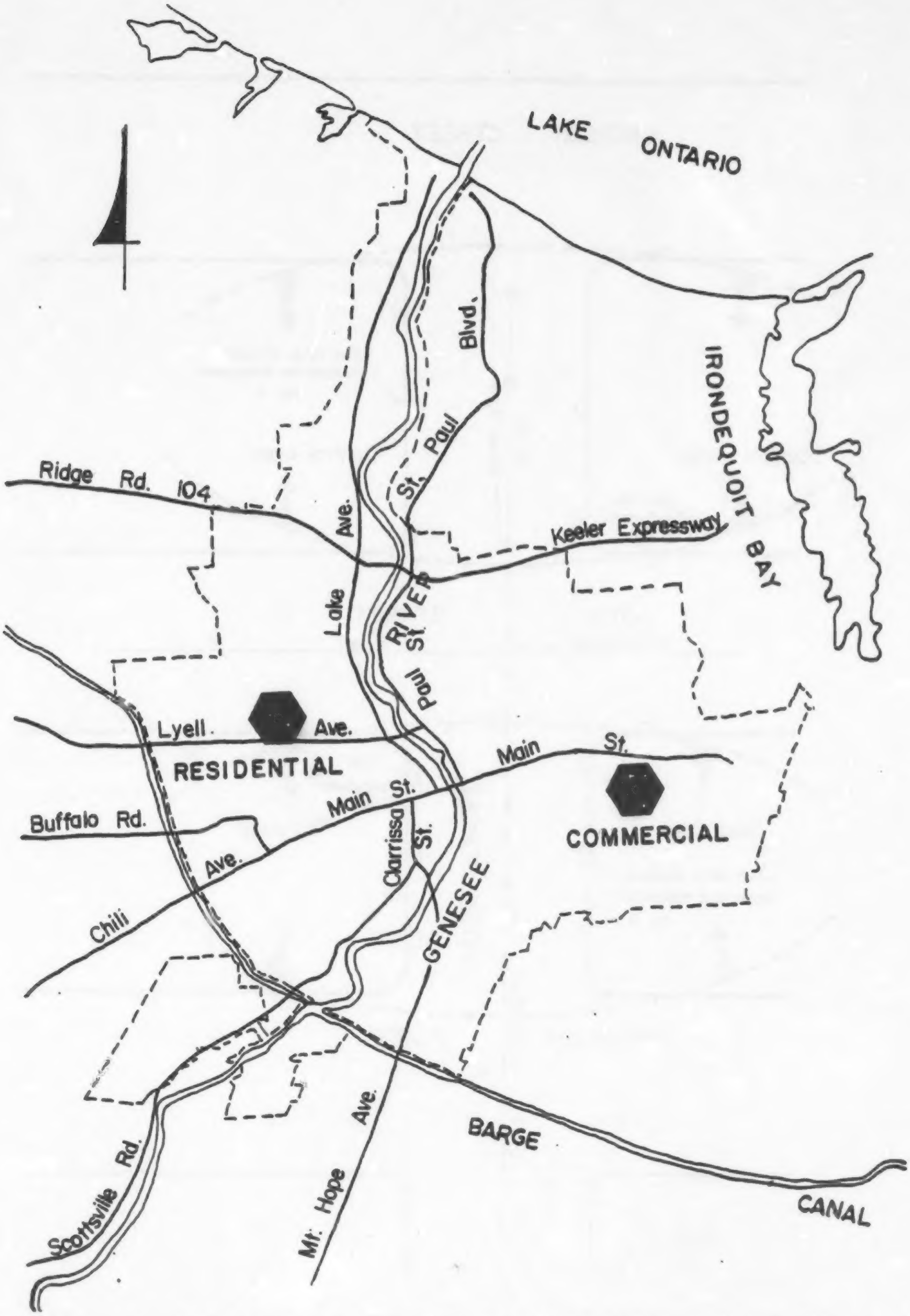

LAKE ONTARIO
IRONDEQUOIT BAY
Ridge Rd. 104
Keeler Expressway
Lake Ave.
St. Paul Blvd.
St. Paul St.
RIVER
GENESEE
Lyell Ave.
RESIDENTIAL
Main St.
COMMERCIAL
Buffalo Rd.
Chili Ave.
Clarissa St.
Scottsville Rd.
Mt. Hope Ave.
BARGE
CANAL

Figure 25. General location map for catchbasin/streetsweeping demonstration study.

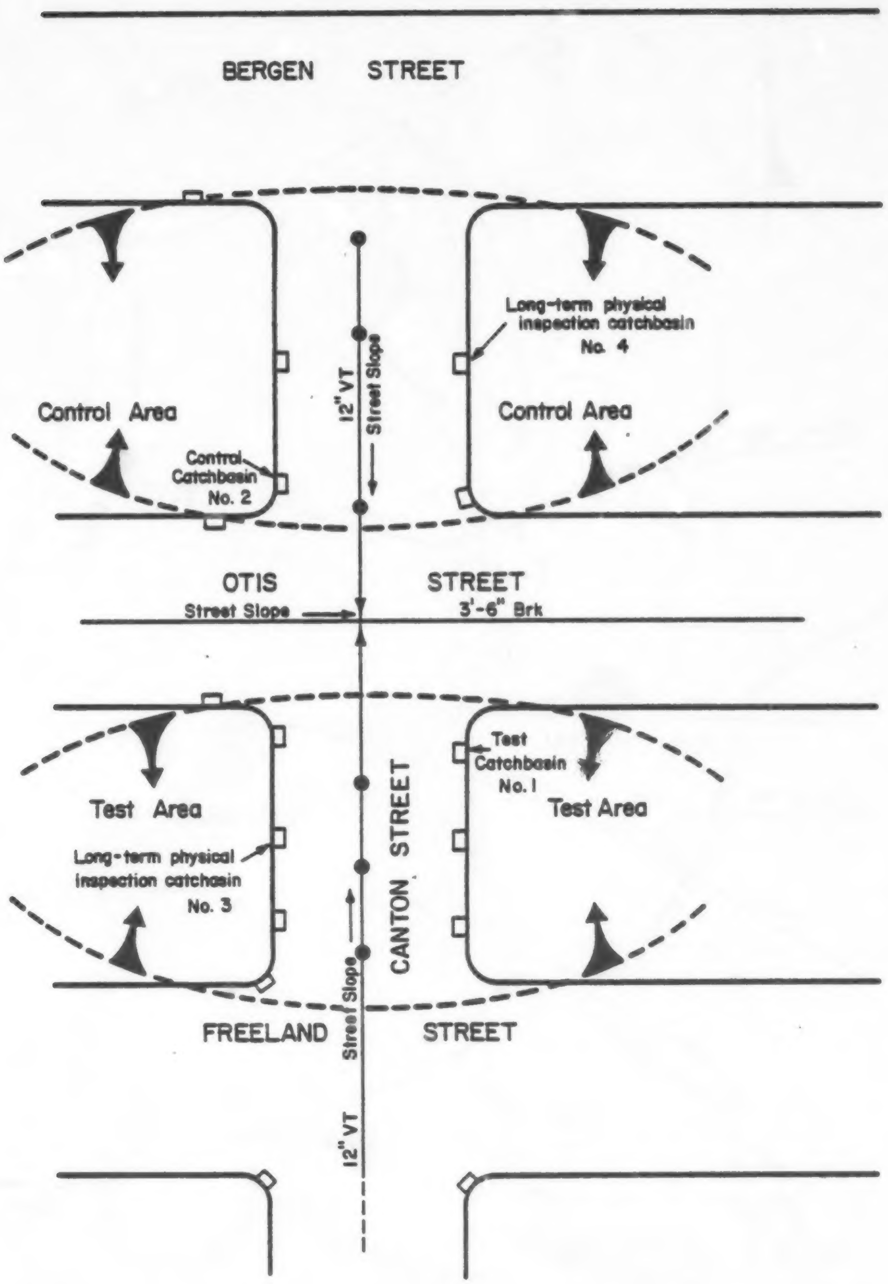

BERGEN STREET
Control Area
Control Catchbasin No. 2
12" VT
Street Slope
Long-term physical inspection catchbasin No. 4
Control Area
OTIS STREET
Street Slope
3'-6" Brk
Test Area
Long-term physical inspection catchbasin No. 3
Test Catchbasin No. 1
Test Area
CANTON STREET
Street Slope
FREELAND STREET
12" VT

Figure 26. Schematic of catchbasin/streetsweeping demonstration site representing a residential area.

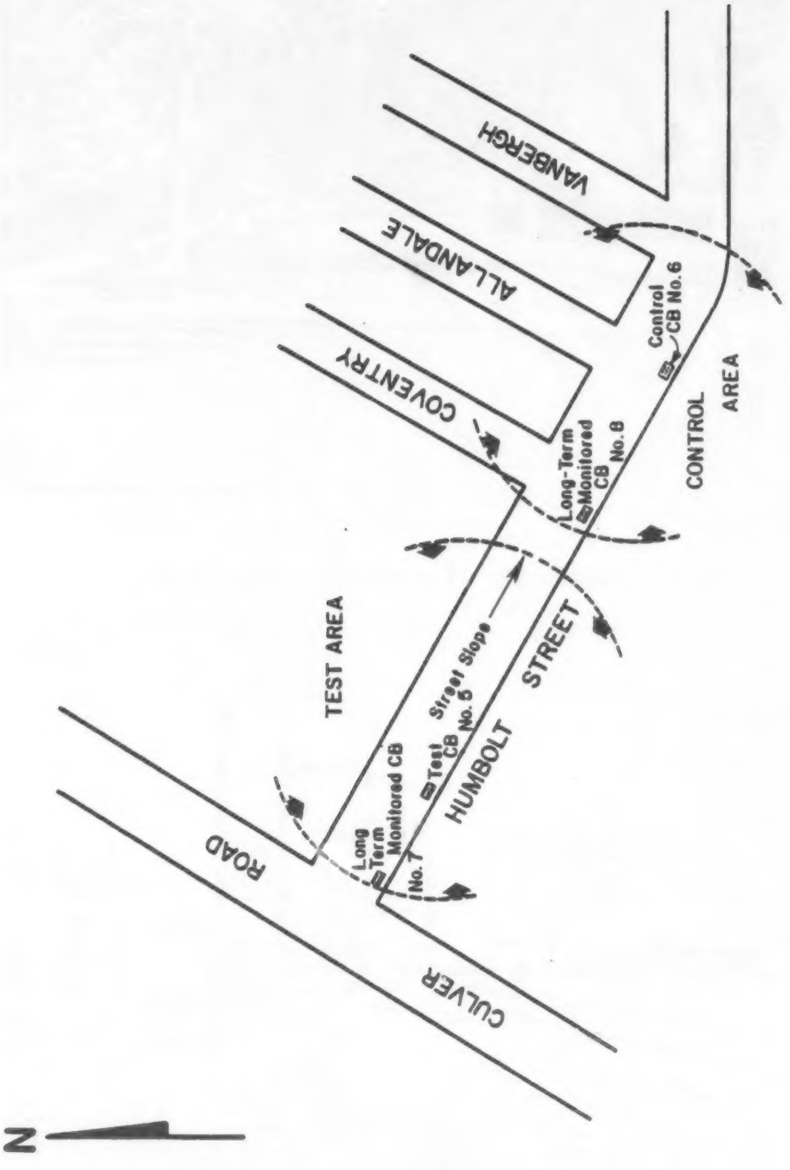

VANBERGH
ALLANDALE
COVENTRY
Control CB No. 6
CONTROL AREA
Long-Term Monitored CB No. 8
TEST AREA
Street Slope
Long Term Monitored CB No. 7
Test CB No. 5
HUMBOLT STREET
ROAD
CULVER
N

Figure 27. Schematic of catchbasin/streetsweeping demonstration site representing a commercial area.

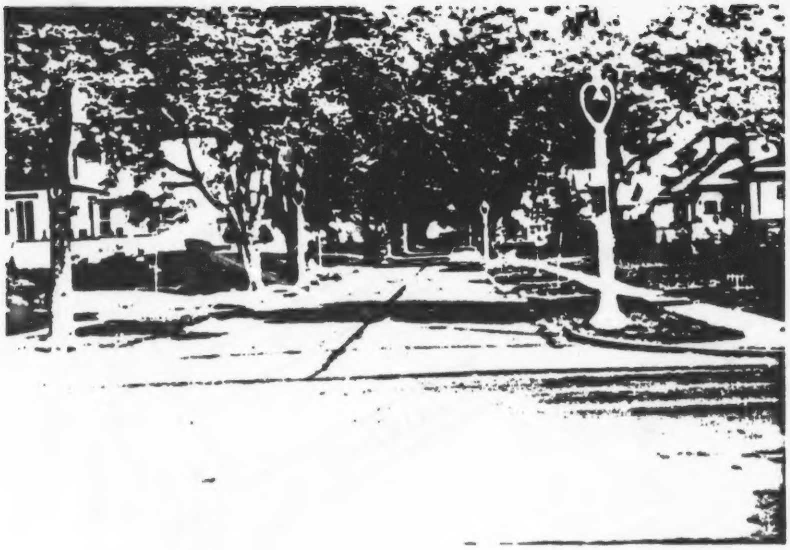


Figure 28. Residential test area.

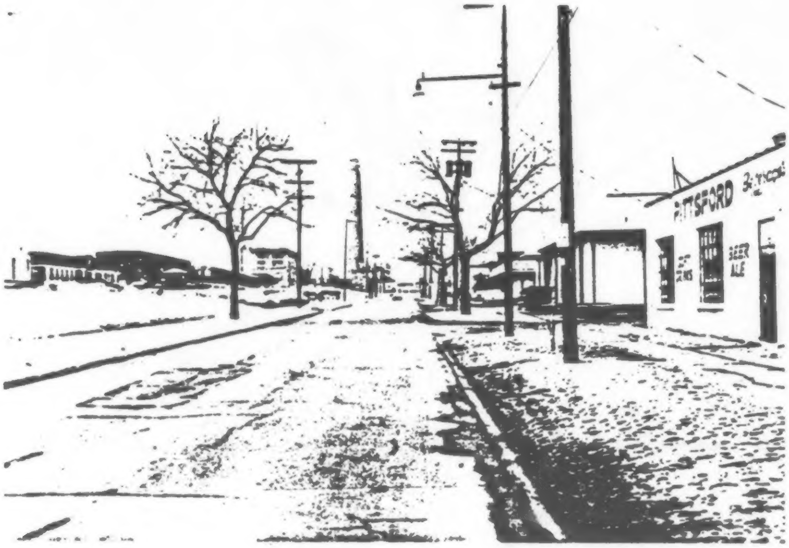

PITTSFORD
BEER
ALE

Figure 29. Commercial test area.

The evaluation program consisted of flow monitoring and sampling of the catchbasins for several storm events under various street sweeping frequencies. Table 28 indicates the evaluation plan that was followed. The plan shows that each test area was to receive twice and eventually three times the normal sweeping frequency. The identified test areas did receive the street sweeping frequency presented in Table 28, whereas, the control areas received the routine sweeping as conducted by the City.

TABLE 28. EVALUATION PLAN FOR CATCHBASIN STUDIES

Date	Week	Street Sweeping Frequency
Apr 16	1	Unaltered
	2	"
	3	"
	4	"
	5	"
	6	"
May 28	1	Increase 100% (double)
	2	"
	3	"
	4	"
June 25	1	Increase 200% (triple)
	2	"
	3	"
	4	"
July 23	1	Unaltered (original)
	2	"
	3	"
	4	"
Aug 20	1	Increase 100% (double)
	2	"
	3	"
	4	"

Flow monitoring from the catchbasins was accomplished by diverting curb flow with aluminum rain gutters into a bucket located in the catchbasin. By timing the interval required to fill the bucket, the rate of inflow into the catchbasin was easily computed. Sampling was conducted by collecting a grab sample of the curb runoff flow at specified intervals.

Results

Results of three monitored storm events are summarized in Table 29. Based on this information the following observations are presented:

- Large variations in the actual pollutant concentrations occurred.

TABLE 29. SUMMARY OF CATCHBASIN MONITORING DATA BY STORM EVENT

Land Use*	Storm Event No.	Rainfall Volume-in.	Volume of RunoffΔ	Test or Control Area+	Sweeping Frequency$	Quality Parameters++			
						BOD	TSS	TKN	Pb
R	1	0.25	620	T	UNALT	31	136	0.1	0.53
R	1	0.25	300	C	UNALT	17	50	1.0	0.34
C	1	0.25	450	T	UNALT	143	100	2.9	0.13
C	1	0.25	500	C	UNALT	215	27	4.2	0.08
	(5/21)								
R	2	0.35	870	T	DOUBLED	16	44	0.1	0.06
R	2	0.35	415	C	UNALT	16	108	0.1	0.13
C	2	0.35	620	T	DOUBLED	20	85	0.3	0.30
C	2	0.35	700	C	UNALT	13	100	0.3	0.29
	(6/28)								
R	3	0.26	650	T	DOUBLED	20	81	N/A	0.08
R	3	0.26	310	C	UNALT	22	138	N/A	0.26
C	3	0.26	460	T	TRIPLED	38	217	N/A	0.87
C	3	0.26	520	C	UNALT	23	164	N/A	0.70

* Land Use - R = Residential, C = Commercial

+ Test/Control Area - T = Test CB, C = Control CB

Δ Runoff Volume is in gallons (gal).

++ Quality Parameters are average values in mg/l.
Total number of samples = 64

$ Normal frequency: residential - once every sixth working day;
commercial - every day

- For Storm Event 1, the street sweeping frequency was the same for both the test and control areas and similar pollutant concentrations were expected. The fact that a large variation was observed was likely a result of other factors not accounted for, such as, volume of traffic, number and location of parked vehicles, and actual street sweeping efficiency.
- For Storm Events 2 and 3, the effect of increased street sweeping was a general decrease in pollutant concentrations in the residential test area as compared to the control area. The impact of increased sweeping in the commercial area could not be determined from these data.
- Doubling of the street sweeping frequency in the residential area resulted in a noticeable decrease in runoff pollutant concentrations; whereas, the effect was much less pronounced for the commercial area. It should be noted that the normal street sweeping frequency in the commercial area of once per day is very high.
- The average pollutant concentrations were significantly higher in the commercial area relative to those measured in the residential area. It was expected that surface loading rates would be higher in the commercial area because of the greater daily traffic volume. The average daily traffic (ADT) for the commercial area was about 13,000 vehicles; whereas, the ADT associated with the residential area was approximately 550.
- Based on these data, for normal street sweeping frequencies the amount of BOD contributed to a catchbasin in the residential area was approximately 0.64 lb/in. of rain; whereas, the commercial area catchbasins contributed about 4.5 times as much. The 0.64 lb BOD/in. of rain appeared to be consistent with the value of 1.07 lb BOD based on 13.5 lb BOD/curb mi. as indicated in an earlier report on catchbasin technology (20). The 0.64 lb BOD/in. value also represented approximately 60% of the pollutant initially on the street surface.

Cost-Effectiveness

The BMP study tended to show that increasing the street sweeping frequency by more than a factor of two essentially had little effect on the amount of pollutants washed into the sewer collection and conveyance system. Because of the limited data base, no quantification of the effectiveness could be made. Qualitatively, however, it was shown that additional street sweeping could reduce the amount of surface contaminants in commercial areas; whereas, the data for the residential area were inconclusive.

Based on results from a recent study on nonpoint pollution abatement through improved street cleaning practices, it was shown that the type and condition of street cleaned affected performance more than the differences in equipment used (19). In addition, there were many factors, some of which could not be readily controlled, such as operator efficiency, that made cost-effective analysis difficult at best. The use of vacuum sweepers could substantially improve the effectiveness of surface pollutant removals but quantifying the improvement extent would be difficult.

Based on previous reports, the average unit cost of street sweeping per curb-mi cleaned varies widely. Estimates from $3.00 to $14.00/curb-mi (1980 dollars) have been reported (19). For the City of Rochester the unit cost was approximately $12.00/curb-mi (1976 dollars). According to the San Jose report, of the total unit cost of street cleaning, approximately 97% is related to maintenance, supplies, disposal of collected material, and labor. Only 3% of the total is related to equipment depreciation. Table 30 presents the costs associated with the street cleaning effort for San Jose, California. A similar cost breakdown was not available from the City of Rochester. It was believed that the relative costs for the various parameters were approximately the same.

Table 30, illustrates the fact that street cleaning is manpower intensive. Because a high proportion of the total cost is labor, additional efforts, in terms of increased street cleaning frequency, will be costly in terms of marginal costs. It appears, therefore, that for increased street cleaning to be effective, substantial surface pollutant removals must be attained. To date, this has not been possible with the present equipment, operator efficiency, and site-specific street conditions.

The data reviewed to date on street sweeping can be best summarized by stating that conventional street cleaning methods are most effective in removing large particulate material, but are unable to remove a significant portion of the pollutants found in the less abundant, smaller particle sizes. Therefore, it will be difficult to meet complete source control objectives unless extra effort is expended. The overall effectiveness of street cleaning would, therefore, depend on the specific service area characteristics and program objectives.

For the City of Rochester, additional effort should not be expended in increased street sweeping frequencies until further data indicate a more definite effectiveness in removal of street surface contaminants. More detailed evaluations are also needed to demonstrate the effectiveness of catchbasin cleaning in removing pollutants from surface runoff. If additional monies became available, the city could concentrate on increasing the street sweeping frequency in those areas identified as high-polluting based on site-specific investigations. It was concluded that a viable cost-effective alternative to CSO pollution abatement was not additional or better street cleaning by the City of Rochester.

POROUS PAVEMENT DEMONSTRATION

General

The earliest applications of porous pavement were primarily for reducing the possibility of vehicle skidding, or hydroplaning, during rainfall. By allowing rainwater to rapidly drain laterally from the roadway surface towards the shoulders, the chance for hydroplaning was greatly reduced. Under

TABLE 30. SAN JOSE ANNUAL STREET CLEANING EFFORT (1976 - 1977)

	COST			LABOR		
	Total Cost ($)	Cost ($/curb-mi cleaned)	Percentage of Total Cost	Total Labor (person-days)	Unit Labor (hr/curb-mi cleaned)	Percentage of Total Cost
Maintenance Supplies[a]	93,000	1.60	12	--	--	--
Operation Supplies[b]	29,000	0.48	3	--	--	--
Disposal	65,000	1.17	8	780	0.12	13
Equipment Depreciation	31,000	0.48	3	--	--	--
Cleaner Operations[d]	326,000	5.76	41	3400	0.50	56
Maintenance Personnel[d]	176,000	3.20	23	1200	0.18	20
Supervisors[d]	80,000	1.44	10	650	0.10	11
Total Annual Costs	$800,000	$14.00	100%	6030 Days	0.90 Hrs	100%
Total Annual Curb-mi Cleaned	55,761 Miles					

[a] Includes gutter and pick-up broom replacement.

[b] Tires, fuel, and oil.

[c] The street cleaners dumped their hopper contents on the streets in interim piles, where they were later removed by front-end loader and dump truck crews. The landfill was located centrally, with maximum distances of about 15 miles from the cleaner routes to the landfill.

[d] These labor costs include administration, warehouse, secretary, and overhead costs.

such applications, only a thin layer of porous material was placed over a conventional, dense asphaltic concrete pavement. Until recently, the use of porous pavement for other purposes was non-existent. The basic concepts involved in utilizing porous pavements for stormwater management systems were initially developed in the early 1970's by the Franklin Institute Research Laboratories in Philadelphia, PA under the sponsorship of the U.S. Environmental Protection Agency (21). The objective of porous pavement was to enhance and use the natural capacity of the soil to adequately absorb stormwater runoff and to replenish groundwater. An ongoing program also investigating this aspect of porous pavement is being conducted by the U.S. Department of Agriculture at the Willow Grove Naval Air Station in Pennsylvania (22).

Although the basic objective in utilizing porous pavement is to reduce surface runoff while providing for groundwater recharge, other applications have gradually developed within recent years. For sufficient groundwater recharge to occur, the underlying soils must have adequate infiltration capacity.

In urban areas it is more likely that porous pavement would be applied over existing pavements to reduce the rate of stormwater runoff; whereas, in new areas with suitable soil conditions, porous pavements can possible provide groundwater recharge potential.

In addition, for structural suitability of the pavement section, the soil must not swell excessively nor heave at low temperatures. Since not all soils throughout the United States can meet all these requirements, hybrid systems have been developed to maximize the use of porous pavement to reduce the impact of stormwater runoff. Thus, porous pavements can be constructed to:

- Retain all precipitation that falls onto the porous surface with no resulting runoff.
- Retain enough precipitation so that runoff is not significantly increased after construction.
- Detain stormwater runoff to reduce peak demands placed on downstream sewer systems.

In addition to the hydrologic aspect of allowing rainwater to infiltrate into the ground by utilizing porous pavements, a significant degree of treatment may also be realized. Combined sewer overflow as well as stormwater contain substantial quantities of undesirable pollutants which may be removed by porous pavements. The inherrent ability of the soil and pavement system to remove many contaminants may mitigate the effect of stormwater infiltration into groundwater supplies.

Benefits can also be attributed to the use of porous pavements other than that of groundwater recharge and reduction in the rate of stormwater runoff. Reductions in storm drainage construction costs, elimination of the need for curbs and gutters, and improved traffic safety resulting from increased skid resistance and improved visibility on wet pavements can also result (21).

Basis of Design

The approach taken for the Rochester BMP program involved the demonstration of the concept of utilizing porous pavement to reduce the rate of stormwater runoff. The effectiveness was determined by comparing runoff rates to those generated from a conventionally paved parking lot utilizing stormwater inlets and surface piping. The ability of porous pavement to retain its structural integrity under varied traffic loadings and freeze-thaw cycles, while maintaining its highly permeable nature, was also evaluated.

Due to the reduction in the rate of stormwater runoff entering a combined sewer system, an appreciable reduction in CSO induced pollution can possibly be realized by utilizing porous pavements. Since most combined sewer systems found in older urban centers are comprised of impervious surfaces with few open areas, use of porous pavements to acheive downstream sewer flow reductions was promising.

Selection of Demonstration Site

Two sites were selected to demonstrate the effectiveness of utilizing porous pavements to reduce the rate of stormwater runoff and to investigate the long-term structural integrity of such pavements under severe environmental conditions. Hydrologic testing of porous pavement was conducted at the Gates-Chili-Ogden Sewage Treatment Plant (GCO STP). This STP, operated by the Monroe County Division of Pure Waters is near the Monroe County Airport. A second demonstration site was at the Monroe County Division of Pure Waters maintenance building on Lake Avenue in the City of Rochester. This site was selected to evaluate the structural integrity and permeability of the porous pavement under conditions of heavy truck traffic. Both test site locations are shown on Figure 30.

These two test areas were selected on the basis of suitable conditions compatible with the overall objectives for each location. To insure that uniform rainfall would occur over the entire hydrologic test site and that the intensity of such rainfall could be accurately measured, an open area free of interfering obstacles such as trees and buildings was required. In addition, the ability to control the number and frequency of vehicles was needed to insure that the pavement was not damaged before sufficient hydrologic testing was conducted. The GCO STP location was ideal in meeting these conditions. The proximity to the U.S. Weather Bureau located at the Monroe County Airport allowed for easy comparison between rainfall data recorded onsite by a local recording gauge and Weather Bureau data.

To properly evaluate the structural integrity of porous pavement, a site was needed that provided for frequent vehicle use. The Lake Avenue site was again ideal. Being the site of the County maintenance department, the parking lot at this location received frequent daily traffic loadings from relatively heavy vehicles such as hydro-flushers and other large trucks. Not only was there frequent traffic but also the vehicles would engage in sharp turning maneuvers, which tested the ability of the porous pavement to resist shearing stresses that could seal the top layer and prevent the rapid downward water movement.

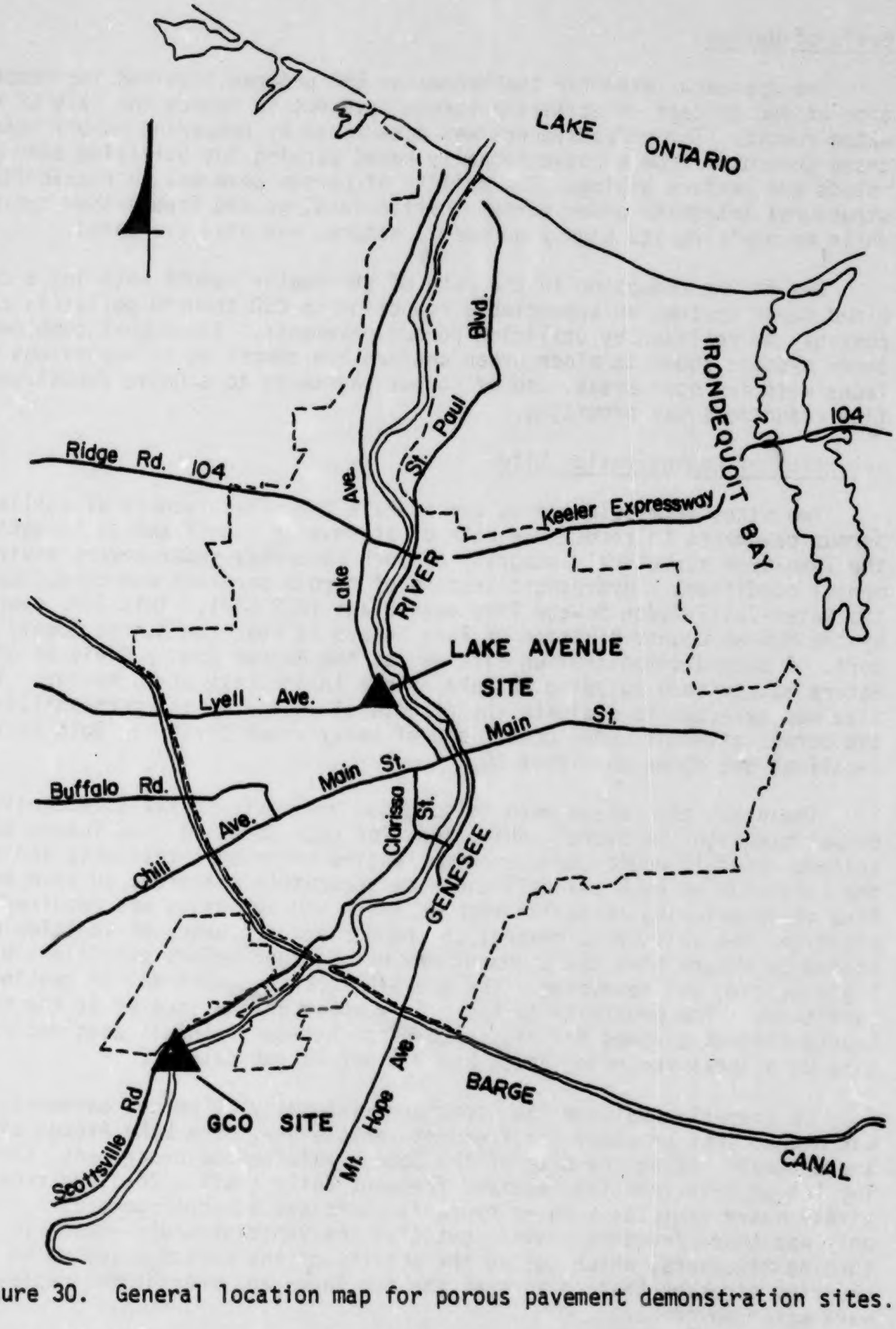

LAKE ONTARIO
IRONDEQUOIT BAY
104
St. Paul Blvd.
Lake Ave.
Ridge Rd. 104
Keeler Expressway
RIVER
LAKE AVENUE SITE
Lyell Ave.
Main St.
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
GENESEE
Mt. Hope Ave.
BARGE
CANAL
GCO SITE
Scottsville Rd

Figure 30. General location map for porous pavement demonstration sites.

The GCO porous pavement test area consisted of two equal areas of pavement, each 100 ft square and comprising 0.23 ac. The test area was constructed of 5 in. of porous asphaltic concrete over a base of crushed stone approximately 9 in. deep. Under the stone base an impermeable asphaltic membrane was applied to prevent collected rainfall stored in the stone base from being transferred to the groundwater.

Under the configuration used in this demonstration, rainfall passed through the porous pavement layers and was temporarily stored in the stone base layer. Reduction in the rate of runoff was accomplished by allowing the storage reservoir to be slowly drained by two 6 in. underdrains placed at the bottom of the stone base. Flow in the underdrains was tributary to a metering pit where the rate of flow was determined by simple volumetric and weir flow calculations.

The control area associated with the GCO test site consisted of a conventionally paved area of equal dimensions but sloped uniformly towards the center, where a standard stormwater inlet intercepted the surface runoff and conveyed it to the same metering pit. Figure 31 shows the general layout of the GCO porous pavement demonstration site. Figure 32 is a photograph of the demonstration site.

The storage reservoir provided by the stone base at the GCO site was sized based on completely storing a 5 yr - 24 hr rainfall for the Rochester area. Technical Paper No. 40 of the U.S. Weather Bureau indicated that a storm of this frequency would contain approximately 3.1 in. of rain. Assuming that the crushed stone base had an inherrent void space of about 40%, a stone base of nearly 8 in. was required. To allow for possible clogging of the stones and to provide for more structural strength of the overall pavement/base configuration due to superimposed traffic loads, 9 in. of stone base was used.

At the Lake Avenue site, a porous pavement structure was placed directly over a stone base without providing an asphaltic membrane beneath the storage reservoir. One underdrain was installed and connected to an existing catchbasin system. At this location rainfall that entered the porous pavement either drained into the one underdrain or entered the groundwater by passing through the soil immediately under the stone base. This type of drainage system was felt to be sufficient to assess the applicability of porous pavement under the general conditions encountered in new residential areas or new parking lots. The ability of the porous pavement to sustain rapid infiltration under traffic loadings was stressed at the Lake Avenue site. Figure 33 shows the general layout for the porous pavement site at Lake Avenue.

Several important observations were noted during the construction of the porous pavement demonstration site. They included the following items which should also be considered in future applications:

- The temperature of the asphalt mix from the batch process to placement and compaction is important. Heating of the aggregate to at least 300^{o}F, prior to adding the asphalt, insures that all moisture within the aggregate will be driven off. Excessive moisture pre-

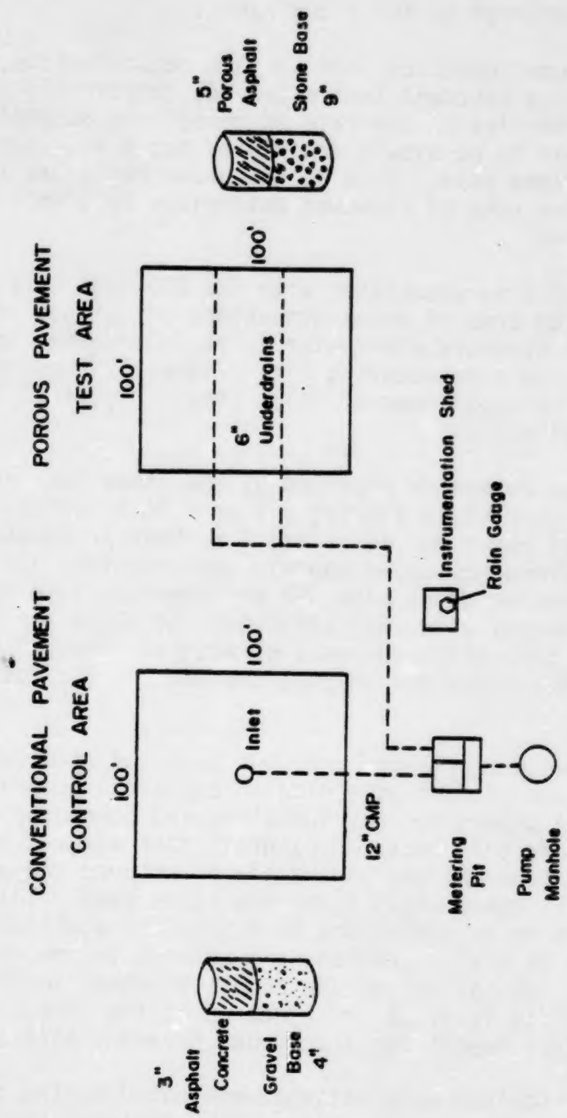

3"
Asphalt
Concrete
Gravel
Base
4"
CONVENTIONAL PAVEMENT
CONTROL AREA
100'
100'
Inlet
12" CMP
Metering
Pit
Pump
Manhole
Rain Gauge
Instrumentation Shed
POROUS PAVEMENT
TEST AREA
100'
100'
6"
Underdrains
5"
Porous
Asphalt
Stone Base
9"

Figure 31. Schematic of porous pavement demonstration site at the GCO treatment plant.

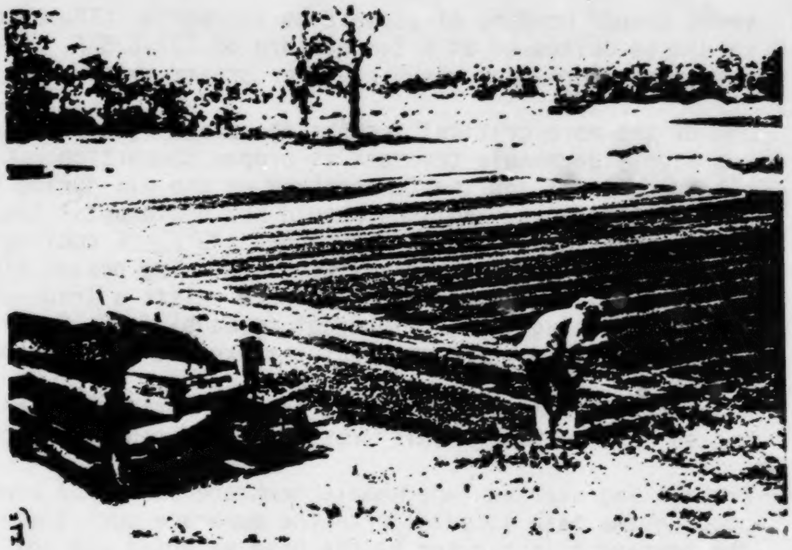


Figure 32. GCO porous pavement demonstration site.

Figure 33. General layout of Lake Avenue porous pavement site.

vents proper bonding of asphalt to aggregate. The asphalt mix should be delivered at a temperature of 225-235°F. A digital readout temperature gauge is useful for proper measurements.

- One of the more critical factors in obtaining a structurally sound but highly permeable pavement is proper compaction. Cooling of the placed material and the temperature of the mix during rolling are important factors. Rolling should not begin until the temperature of the surface and mid-pack is about 170°F. A cooling rate of approximately 60°F/hr appears adequate. Two passes with a steel 8-10 ton roller is sufficient. There exists a trade-off between structural integrity and inherent permeability. Greater compaction results in stronger pavement at the expense of lower permeability.

- An asphaltic membrane, if used, is much better applied in several passes that overlap. This provides for a tight impermeable seal.

- Backfilling over an impermeable membrane should be accomplished by pushing the base material onto the membrane such that the weight of the equipment is carried by the base material and not by the membrane.

- Prior to applying an asphaltic membrane, a smooth, uniform subbase surface should be provided.

- If the base material consists of stone, the maximum lift should be 8 to 9 in. to protect an asphaltic membrane. This layer should be thoroughly rolled to provide maximum compaction. It cannot be over compacted.

- Placing the open graded porous material on a wet asphaltic treated porous material layer should be avoided to prevent rapid cooling of the applied asphalt, which would result in aggregate binding and decreased permeability.

- Additional rolling with a light roller can be made to remove any small ridges in the pavement created during the compaction with the heavy roller.

Monitoring Program

Hydrologic Testing--

The results of the hydrologic testing of porous pavement at the GCO demonstration site indicated that the type of porous pavement system utilizing an impermeable membrane and underdrains could substantially reduce the peak runoff rate relative to that from a conventionally paved area with stormwater inlets. A monitoring program was conducted from September, 1979 through August, 1980. Winter months were not included because of snow and ice buildup in the metering pit where flow measurements were taken. Comparisons were made as to the rainfall recorded by the local rain gauge at the demonstration site and that recorded by the U.S. Weather Bureau located only a short distance to the east.

Tables 31 and 32 present the monitored rain and flow data for 1979 and 1980, respectively. With respect to rainfall, very close correlation (r=0.97) between the total rain recorded at the site and that reported by the Weather Bureau was obtained. In terms of rainfall intensity, the values were different because of the time interval used to calculate the intensity. The interval used for the Weather Bureau data was 1 hr; whereas, the maximum slope of the actual rainfall hyetograph, as measured by the local rain gauge, was used for site-specific conditions. The rain gauge analysis indicated that rainfall at the Weather Bureau location and at the demonstration site was generally similar, but large variations can occur.

Tables 31 and 32 indicate that the peak runoff rate from the porous pavement was significantly lower than that from the control area. The percent peak flowrate reduction varied from 18 to 100 depending on the specific rainfall event. The average reduction was 76%. To illustrate the effect of reducing the runoff hydrograph, Figure 34 is presented. As shown, the entire hydrograph for the porous pavement test area was substantially lower that that for the control area. In addition, a lag time of approximately 15 min was observed; that is, outflow from the porous pavement underdrain system did not start until 15 min after the start of rainfall. There was essentially no lag time associated with the conventional pavement because of the rapid surface runoff and the minimal travel distances involved.

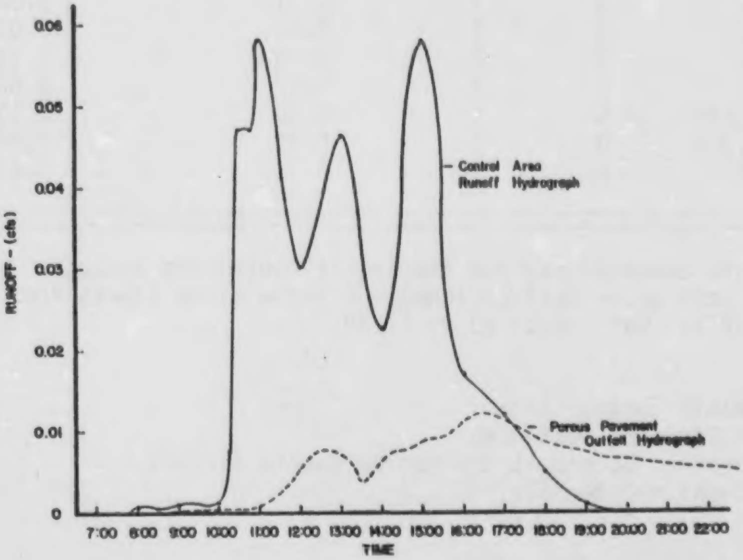


Figure 34. Runoff hydrographs for 4/28/80 storm event.

TABLE 31. 1979 POROUS PAVEMENT DATA BASE

Date	Peak Q - cfs			Rainfall			
	CB	PP/CB	PP	Total - In.		Peak - In./Hr	
				WB	GCO	WB	GCO
9/02	0.41	0.11	0.044	0.57	0.69	0.31	3.00
9/03	0.02	0	T	0	0.06	0	3.00
9/04	0.01	0	T	0	0.02	0	0.01
9/05	0.02	0	T	0	0.05	0	2.55
9/06	0.02	0.50	0.01	0.50	0.48	0.14	0.11
9/10	0.02	0	T	0.33	0.33	0.12	0.32
9/14	2.50*	*	5.08	3.54	N/A Δ	0.49	N/A Δ
9/18	0.21	0.07	0.015	0.21		0.20	
9/28	T	0	T	0.15		0.04	
10/03	0.08	0.05	0.004	0.18		0.12	
10/05	0.063	0.38	0.024	0.84		0.10	
10/06	T	0	T	0.03		0.01	
10/07	T	0	T	0.02		0.02	
10/08	T	0	T	0.09		0.04	
10/09	0.036	0.03	0.009	0.22		0.07	
10/10	T	0	T	0.02		0.02	
10/12	T	0	T	0.21		0.04	
10/13	T	0	T	0.03		0.03	
10/14	T	0	T	0.05		0.03	
10/17	T	0	T	0.07		0.06	
10/20	0.086	0	T	0.30		0.11	
10/23	0.008	0	T	0.29		0.07	
10/24	T	0	T	0.01		0.01	

* Flowrate exceeded maximum reading of monitoring device

Δ Local rain gauge malfunctioned for those given storms from 9/14 during 1979. Unit repaired by 11/79.

CB = Catchbasin Control Area
PP = Porous Pavement Test Area
WB = U.S. Weather Bureau at the Monroe County Airport
GCO = Gates-Chile-Ogden STP

TABLE 32. 1980 POROUS PAVEMENT DATA BASE

Date	Peak Q, cfs			WB Rain Data		GCO Rain Data	
	CB	PP/CB	PP	Total in.	Peak in./hr	Total in.	Peak in./hr
5/13	.0089	0.054	.0048	.31	.08	.37	.30
5/17-18	.14	0.64	.09	.48	.12	.59	.24
5/31	.497	0.80	.40	.47	.25	.41	1.60
6/1	.88	0.45	.40	.40	.32	.49	1.08
6/6	.497	-	-	2.12	1.97	1.92	3.00
6/7-8	.497	-	.497	2.16	1.05	2.46	6.60
6/15	.0046	0.13	.0006	.15	.07	.15	1.20
6/19-20	.203	0.14	.028	.71	.24	0.99	4.50
6/26	.226	0.27	.061	.28	.20	.30	1.20
6/28	.102	-0-	-0-	.48	.25	.53	0.60
7/2	.088	0.36	.032	.19	.16	.20	0.60
7/5	-0-	-	.12	.04	.02	.07	0.60
7/15	.07	-0-	-0-	.07	.07	.11	.40
7/17	.497	0.76	.38	.10	.07	.40	1.54
7/20	.003	-0-	-0-	.06	.03	.07	1.20
7/22	.497	-	.497	1.02	.45	1.00	1.39
7/27	.226	0.11	.025	.16	.11	.25	1.28
7/29	.041	-0-	-0-	.02	.01	.09	0.96
8/2	.280	0.82	.23	.13	.06	.21	2.60
	.497	-	.497	.72	.72	.56	2.80
8/5-6	.242	0.50	.12	.60	.26	.60	2.20
8/15	.497	-	.95	1.24	.71	.89	1.48

The data also indicated that the total volume of runoff was approximately the same for both the test and control areas. Because of the larger flowrates associated with the conventional pavement, a mass rainfall-runoff balance was easily conducted. Due to the configuration of the metering pit, the manner in which flows were measured, and the low flowrates from the porous pavement, an accurate mass balance, similar to that for the control area, could not be established. However, Figure 34 does illustrate the flow attenuation effect of the underdrain system associated with the porous pavement test area. Flow occurred at a much lower rate but continued over a longer period of time relative to the hydrograph for the control area.

At the Lake Avenue porous pavement demonstration site no actual flow measurements were taken. The basic purpose of this site, in addition to structural evaluations, was to evaluate direct infiltration of incipient rainfall into the subbase. It was observed that all rainfall passed through the porous pavement structure.

Granular material was stored immediately off the pavement area. During heavy rainfall, washout of this material onto portions of the porous pavement occurred. This material was largely responsible for partial clogging of the porous pavement. Protection from erosion of adjacent ground surfaces should be provided to maintain porous pavement permeability. Field observations indicated that material carried onto the pavement from vehicular traffic was not a major contributor to clogging problems.

Structural Integrity--

During 1979, traffic was not allowed on the porous pavement area at the GCO demonstration site. Heavy infrequent truck traffic was allowed on the test area during most of 1980. No observable structural degradation of the porous pavement was noted. In addition, no problems resulting from actual freeze-thaw conditions were observed.

It was also generally observed that snow and ice did not accumulate for long periods of time on the porous pavement relative to the conventional pavement. Melted snow and rainfall continued to pass through the porous pavement during the winter months.

At the Lake Avenue site, a large daily volume of heavy vehicles passed over the porous pavement parking area. After 18 months of use, no noticeable structural degradation was observed.

Permeability Testing--

Previous permeability testing of porous pavement using simplified field techniques indicated that pavement infiltration rates as high as 1000 in./hr can be achieved (21). To more accurately define the actual inflow potential of porous pavement, 4 in. diameter cores of the pavement were obtained from both the GCO and Lake Avenue demonstration sites. Laboratory permeability testing was conducted during 1979 which indicated that the inflow rate varied from 1980 in./hr at the GCO site to 170 in./hr and 43 in./hr at the Lake Avenue site. The two values associated with the Lake Avenue site represented test results for cores obtained from clean and dirty areas, respectively. The clean area represented that portion of the test pavement that received little or no traffic; whereas, the dirty area represented a portion that received heavy traffic loading and a heavy application of soil particles washed onto the pavement.

Permeability testing conducted during 1980 indicated that the inflow potential for all three areas remained high, although noticeably reduced from the initial test values. These test results ranged from 540 in./hr for the GCO site to 160 in./hr and 27 in./hr for the Lake Avenue site.

To determine the effect of abrasives added to roadway surfaces during the winter months on the inherent permeability of porous pavements, a simple test procedure was established. The testing consisted of the addition of a predetermined volume of beach sand to a pavement test core after which a falling head permeability test was performed. According to a report published in 1976 approximately 1.0 lb abrasives/yd^2/application are used by most highway departments for deicing purposes (23). The test procedure attempted to approximate actual application rates, the effect of vehicle loading, and the effect of alternating periods of snowmelt and dry days. Although many cities in the Northeast do not use abrasive materials but rather use calcium or sodium chloride, clogging of porous pavements resulting from applications of salt was anticipated to be minimal at best. Therefore, testing of the use of abrasives was initially conducted, followed by testing with road salt. Testing with salt in conjunction with freeze/thaw testing to determine structural integrity was also conducted.

The basic equation used in determining permeability was as follows (24):

$$K = \frac{aL}{At} \ln \frac{ho}{h_1}$$

where K = permeability coefficient - in./hr

a = x-sectional area of standpipe - in^2

A = x-sectional area of sample - $in.^2$

L = length of sample - in.

t = time - hr

h_0, h_1 = original and final hydraulic heads, respectively - in.

In the tests conducted, the areas of the sample and of the standpipe were identical; therefore, the equation was reduced to:

$$K = \frac{L}{t} \ln \frac{ho}{h_1}$$

That is, the permeability rate is directly related to the length of the test section, inversely proportional to the time it takes for the water in the standpipe to fall a specific distance, and directly related to the log of the ratio of initial and final hydraulic heads.

To determine the decrease in permeability of porous pavements resulting from the application of abrasive materials, a predetermined volume of sand was added to the top of a core sample and a falling head permeability test was then conducted. To approximate a similar road application rate the following procedure was adopted in the laboratory test. Assuming that 1.0 lb abrasive material/yd^2 is applied after each snowfall, and that during a severe winter approximately 50 sanding operations are conducted, a scaled-down amount 1.0 $in.^3$ of sand was added to the test core. To simulate the cumulative effect of the annual addition of sand to roadways, increments of 1 $in.^3$ of sand were applied and permeability tests were conducted after each such application. Figure 35 presents the effect of sand addition on the permeability rate. A marked decrease in the K value with initial applications of sand and a rapid leveling off after successive loadings was observed. It is important to note that even after six such applications, the permeability rate of 27 in./hr still represented a very high rate. Rainfall intensities of this magnitude are never encountered in the Rochester area. Instantaneous peak intensities may approach this value but their occurrence would be very rare.

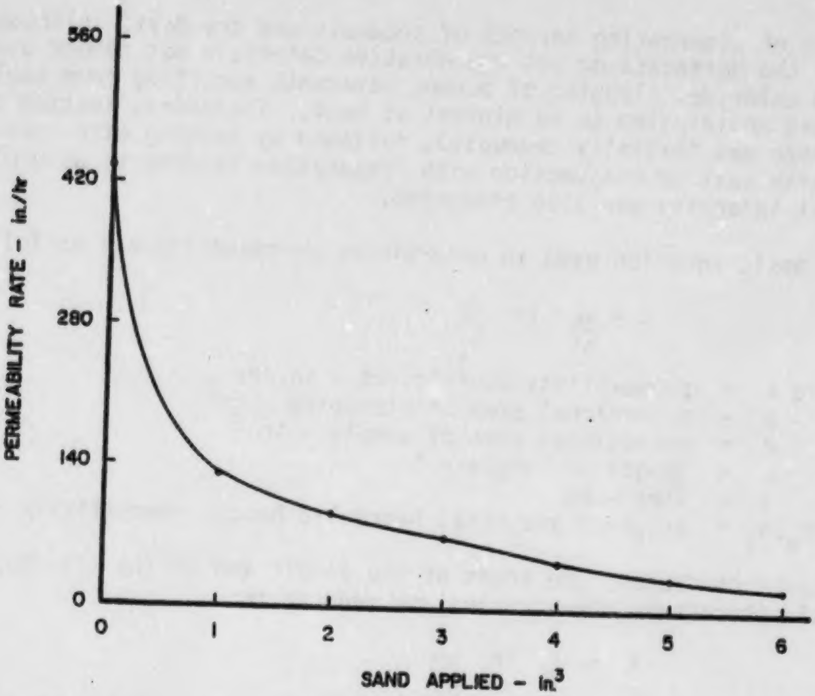


Figure 35. Porous pavement permeability testing with sand addition.

Soils Suitability for Porous Pavement

All pavements require support from the subgrade to prevent excessive pavement deformation and break-up. The subgrade generally does not have adequate support and resistance to water and frost; therefore, a more adequate base is required between the pavement and soil subgrade.

The following factors, relating to the subgrade, should be considered in the design of porous pavements:

- Supporting capability of the soil subgrade
- Water storage capacity
- Frost penetration

The load bearing capacity of the soil subgrade is generally measured by the California Bearing Ratio (CBR) technique. This technique consists of measuring the load required to drive a piston of a standard size into a soil sample at a given rate. The CBR is the load in lb/in.^{2} required to drive the piston a distance of 0.1 in. or 0.2 in. and is expressed as a percentage of the load required to drive the same piston an equal distance into a standard sample of crushed stone.

The water storage capacity of porous pavement depends on the soil's capacity to allow infiltration of water. Subgrades with low infiltration rates (0.01 ft/day) require greater base depths to allow rainfall to infiltrate in a reasonable time period, which was arbitrarily selected as 10 days (24). Fine soils, such as silt and clay, generally have infiltration rates so low that they are not suitable as subgrade materials for porous pavements. It is necessary to replace any unsuitable soil with more permeable soil to provide a higher storage capacity and infiltration rate.

Climates where frost penetrates the pavement structure and subgrade, may cause structural damage to the pavement. Based on a 10 yr return frequency, frost penetration in the eastern United States varies from 12 in. in northern Tennessee to about 100 in. in Maine. Additional base materials are required in areas where the subgrade is susceptible to frost heave. Those soils comprising less than 40 percent by weight of silt and clay are considered frost resistant.

Soil Type Favorable for Porous Pavement--

The ideal soil subgrade for porous pavement applications would have the following characteristics:

- Adequate load-bearing capacity when both wet and dry
- Well-drained
- High permeability
- High porosity

The permeability of the soil should be greater than 0.01 in./hr if infiltration is to occur at a reasonable rate. A sandy silt soil layer well above the water table would likely have the above qualities. This type soil has particles ranging in size from sand to clay. However, the clay content must be low enough to allow the stated minimum infiltration rate. A small amount of asphalt may be added to a highly sandy soil to improve cohesion between the sand particles, thereby increasing the load bearing capacity of the soil. Soils of high clay content are unacceptable, since the low porosity severely limits infiltration and the soils may absorb more water than necessary to fill the voids. The latter situation results in swelling and a corresponding decrease in load bearing capacity. In certain situations, a clayey soil may be treated by adding cement or lime resulting in a more porous mass.

Monroe County Soils--

As stated above, a sandy silt soil above the water table with a permeability greater than 0.01 in./hr is ideal for porous pavement subgrades. The soils in Monroe County are highly variable and include glacial till, shale, and limestone. These soils generally have highly adequate infiltration rates to allow their use as subgrade materials for porous pavements. In Monroe County the seasonal high water table is generally 0-2 ft below the surface (25). The water table could at times severely limit infiltration through porous pavements into the subgrade, thereby resulting in surface ponding.

The maximum freezing depth below the surface is also important for proper operation of porous pavements. Frost heaving and ultimate cracking of the pavement may occur if adequate considerations are not given to this parameter. Engineering projects generally allow for a maximum freezing depth of 42 in. in upstate New York including Monroe County. Therefore, the undisturbed subgrade soil should be a minimum of 42 in. below the surface. The pavement and base would lay above the subgrade. Assuming that 5 in. of porous pavement is used, about 3 ft of base would be required to prevent freeze-thaw damage to the pavement.

A review of soils found in Monroe County indicated that there are areas favorable for porous pavement applications. Final determination of favorable areas must be evaluated on a site-specific basis. Important parameters that should be investigated include: soil permeability, particle size distribution, soil bearing capacity, depth to water table, and maximum depth of frost penetration.

INLET CONTROL CONCEPTS

General

Over the years, surcharged storm and combined sewer systems and surface flooding have caused considerable property damage. In an effort to alleviate the effects of excessive runoff, alternative solutions have been implemented in many instances. Traditionally, areas frequently subjected to flooding have installed new stormwater relief sewers to convey more of the runoff away from homes at a faster rate. The installation of backwater valves and the construction of detention ponds have also helped to alleviate the flooding potential. Regardless of the type of remedial action taken, in many areas, these "flood-proofing" measures have only caused problems to occur elsewhere in the drainage basin. New relief sewers normally are very expensive, both in terms of construction and disruption to existing neighborhoods.

Many large urban centers have initiated costly structurally-intensive construction programs to solve pollution problems resulting from combined sewer overflow and stormwater discharges to receiving water bodies. Many of these programs are also intended to relieve the individual homeowner of basement back-ups and widespread surface flooding. A less costly and positive partial solution to certain drainage relief problems has recently been introduced in the United States and should be considered as an alternative solution to specific drainage problems (26). The basic concept of inlet control falls under the general category of Best Management Practices.

By necessity, any sewer conveyance system does not have sufficient capacity to convey all of the stormwater runoff resulting from all possible rainfall events. To provide for every possible event would not only be prohibitively expensive but, in many instances, would result in moving pollution or flooding problems downstream potentially inducing more serious damage. The past policy of removing stormwater runoff as quickly as possible has resulted in damage to structures downstream as well as to the environment as a whole. By providing inlet control such that the rate of stormwater inflow does not exceed the capacity of the existing collection system, many problems such as basement back-ups, shock pollutant loadings to treatment facilities resulting from the first-flush phenomenon and pollution from CSO discharges can be minimized or eliminated. However, stormwater inlet control will result in a greater level of surface flooding. In most instances, however, stormwater temporarily detained in the streets will result in only minor problems of inconvenience; whereas, stormwater allowed to enter the sewer system at uncontrolled rates can result in significant damage from sewer surcharge and downstream flooding.

One type of inlet control method recently introduced into the United States is a device known as the Hydro-Brake. This flow regulator was invented in Denmark about 15 years ago but has only been marketed in North America since 1975, by Hydro Storm Sewage Corporation. The Hydro-Brake is a flow controller constructed of non-corrosive stainless steel, is self-regulating, contains no moving parts thereby requiring little or no maintenance, and requires no power to operate. The device relies on the static head of stored water to create its own energy to retard flow. The movement of water through the Hydro-Brake involves a swirl pattern action which dissipates energy to control the rate of discharge. Figure 36 presents a schematic of a Hydro-Brake regulator.

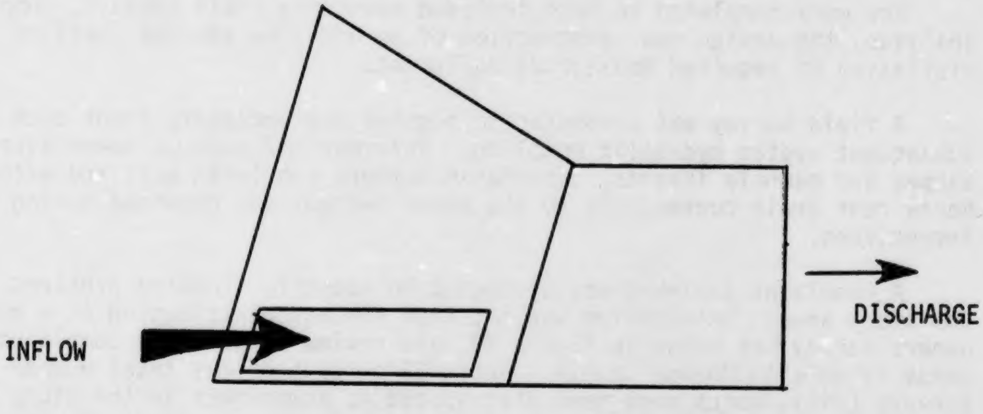


Figure 36. Schematic of a Hydro-Brake regulator.

Santee Hydro-Brake Demonstration Program Description

The concept of inlet control to reduce downstream sewer system surcharging and resultant CSO discharges is being demonstrated in the Santee Drainage Area within the City of Rochester.

The demonstration study area is located on the west side of the city and comprises 35 acres of primarily single family residential houses. The entire area is served by a combined sewer system which is tributary to the West Side Trunk Sewer. There are about 250 houses and 8 commercial establishments within the drainage area, many of which have roof leader connections to the combined sewer system. Sixty-two catchbasins within the area are presently connected directly to the combined sewer system. A schematic of the demonstration site is presented in Figure 37.

The demonstration study consisted of two separate and independent investigations. First, the concept of off-line storage detention with controlled release from the tank by a Hydro-Brake regulator was to be demonstrated. Secondly, the concept of utilizing more surface/street ponding of stormwater to reduce the inflow to an existing combined sewer system was to be evaluated. Each option, or a combination of the two, offered a viable methodology to reduce the rate of inflow into the sewer system through use of flow restrictors. Such flow restrictors or regulators could be Hydro-Brakes or defined orifices to throttle runoff inflow. One advantage of the Hydro-Brake unit over an orifice was that the head-discharge relationship could be specified and controlled with a Hydro-Brake such that discharge would be essentially independent of hydraulic head conditions acting on the device.

The work completed to date included necessary field surveys, complaint analyses, the design and construction of an off-line storage facility and installation of required monitoring equipment.

A field survey was conducted to acquire the necessary input data for subsequent system hydraulic modeling. Information, such as sewer sizes, slopes and manhole inverts, catchbasin numbers and locations, and extent of house roof drain connections to the sewer system, was obtained during field inspections.

A complaint analysis was conducted to identify flooding problems within the study area. Information was obtained through distribution of a homeowners survey, as shown in Figure 38, and review of previous complaint records filed with Monroe County. About 25 percent of the total number of surveys (241), which were hand distributed to homeowners in the study area, were returned. Table 33 presents the street location, number of responses, and the number of responses which indicated basement flooding problems.

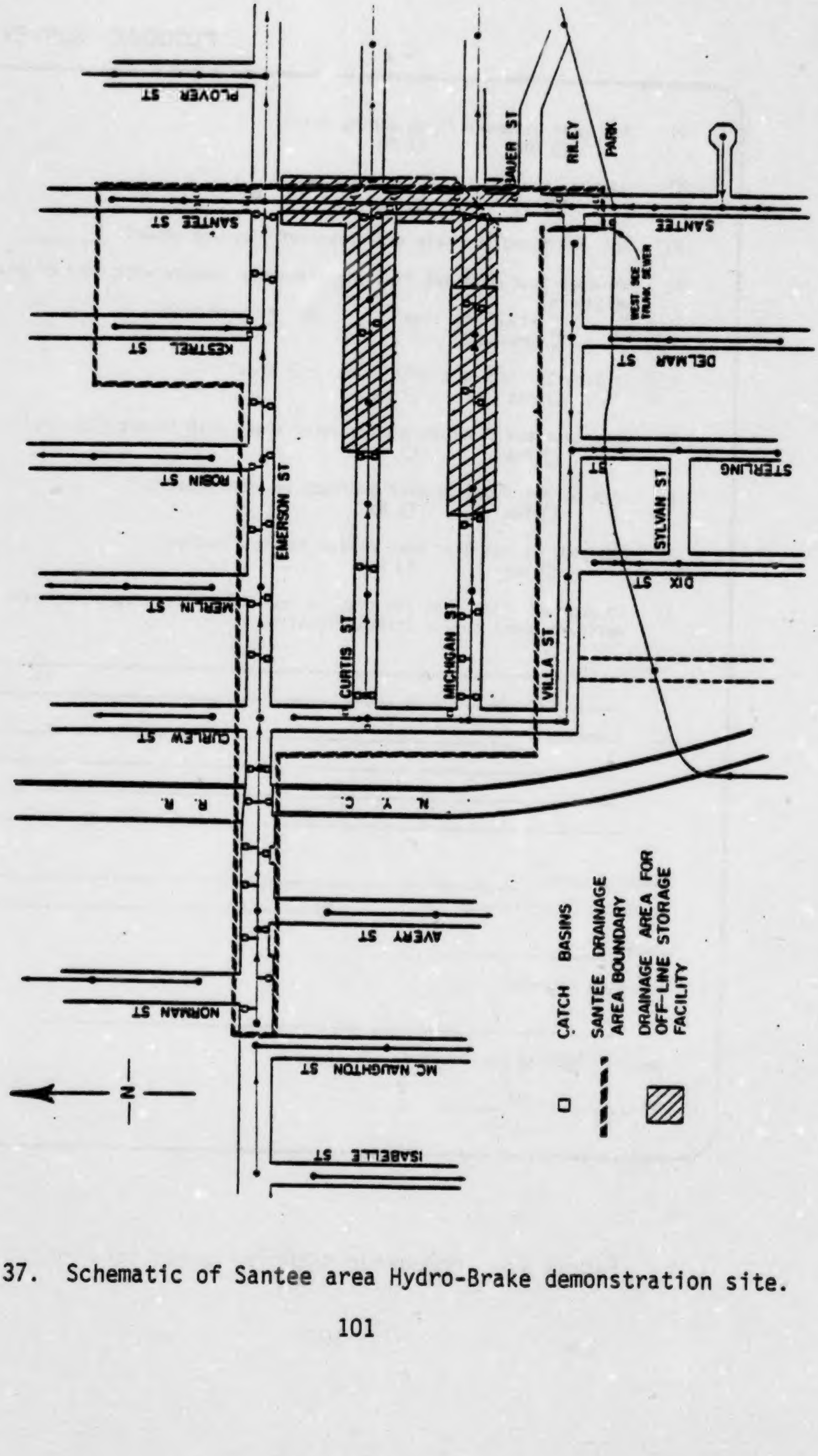
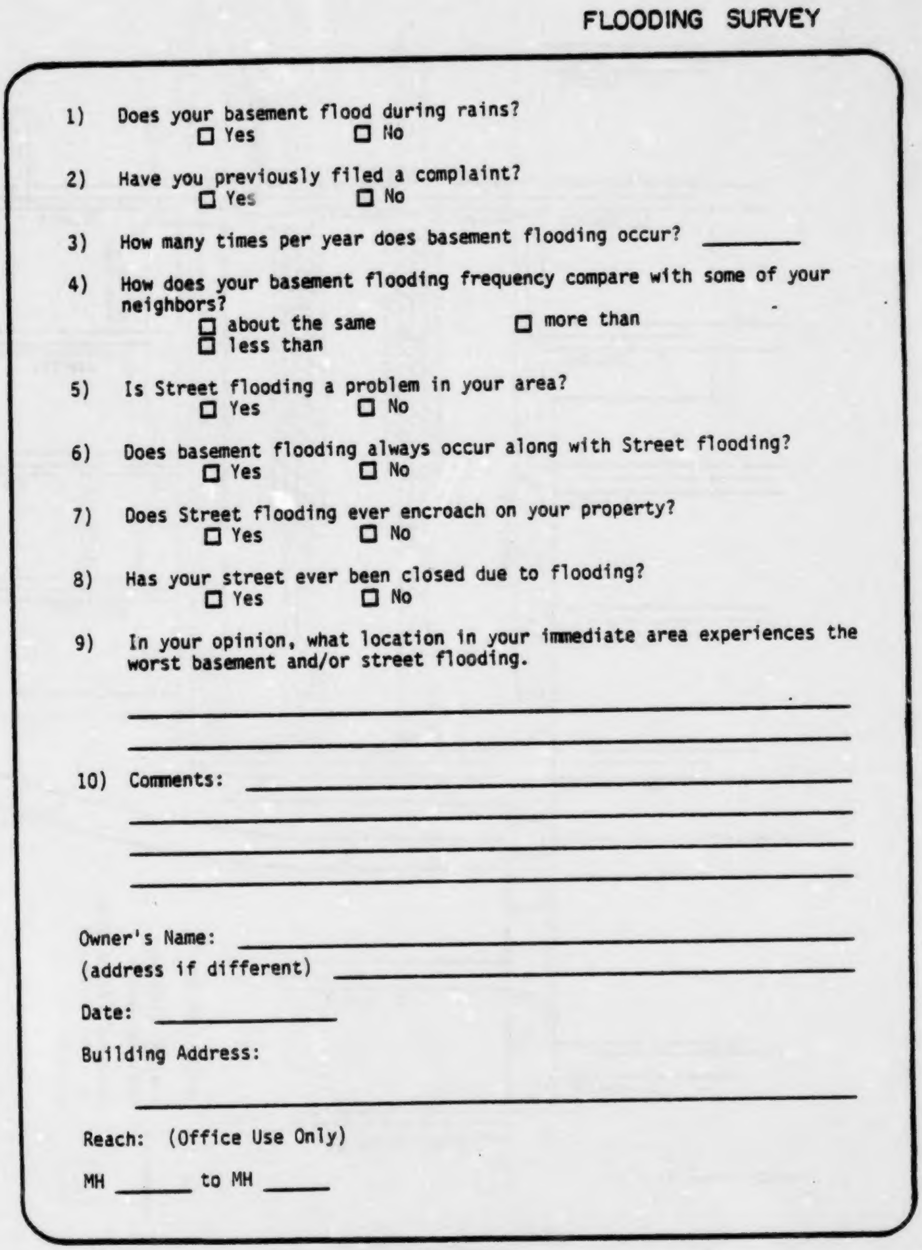

PLOVER ST
SANTEE ST
SANTEE
BAUER ST
RILEY
PARK
WEST SIDE TRUNK SEWER
KESTREL ST
DELMAR ST
ROBIN ST
EMERSON ST
STERLING
SYLVAN ST
ST
DIX ST
MERLIN ST
CURTIS ST
MICHIGAN ST
VILLA ST
CURLEW ST
R. R.
N. Y. C.
AVERY ST
NORMAN ST
MC NAUGHTON ST
ISABELLE ST
N
CATCH BASINS
SANTEE DRAINAGE AREA BOUNDARY
DRAINAGE AREA FOR OFF-LINE STORAGE FACILITY

Figure 37. Schematic of Santee area Hydro-Brake demonstration site.



FLOODING SURVEY

1) Does your basement flood during rains?
☐ Yes ☐ No

2) Have you previously filed a complaint?
☐ Yes ☐ No

3) How many times per year does basement flooding occur? ________

4) How does your basement flooding frequency compare with some of your neighbors?
☐ about the same ☐ more than
☐ less than

5) Is Street flooding a problem in your area?
☐ Yes ☐ No

6) Does basement flooding always occur along with Street flooding?
☐ Yes ☐ No

7) Does Street flooding ever encroach on your property?
☐ Yes ☐ No

8) Has your street ever been closed due to flooding?
☐ Yes ☐ No

9) In your opinion, what location in your immediate area experiences the worst basement and/or street flooding.
____
____

10) Comments: ________________________________
____
____
____

Owner's Name: ________________________________
(address if different) ________________________

Date: ____________

Building Address:
____

Reach: (Office Use Only)

MH ______ to MH _____

Figure 38. Homeowner's survey questionnaire.

TABLE 33. SANTEE HOMEOWNER SURVEY RESULTS*

Street	No. of Responses Received	No. of Responses w/ Basement Flooding
Michigan	14	6
Curtis	16	2
Emerson	14	3
Kestrel	3	0
Curlew	3	0
Santee	6	0

* Based on a 25% survey response

A review of the Monroe County complaint records indicated several occurrences of basement flooding in the Santee Drainage Area over the last three years. The complaint records, however, were so brief that it was difficult to determine the impact that surcharging of the combined sewer system may have had on basement flooding.

Off-Line Storage Facilities--

The purpose of the off-line storage tank with Hydro-Brake regulated outflow was to demonstrate the effectiveness of off-line storage on the reduction of downstream sewer surcharge potential. The installation of small Hydro-Brakes (flow restrictors) in catchbasins would serve to restrict or limit the rate of stormwater runoff into the sewer system; whereas, use of an off-line storage tank with regulated outflow would throttle downstream sewer flows after stormwater runoff had entered the collection system.

Figure 39 presents a layout of the storage facility area including number and location of the catchbasins which were constructed tributary to the storage tank, the piping route to the storage tank, and the outlet pipe to the combined sewer system at Santee and Villa Streets. Discharge from the detention tank is controlled by a Hydro-Brake regulator. The maximum discharge rate will be approximately 0.50 cfs (0.32 mgd). In order to restrict the flow in the outlet pipe from the existing storage tank, until a Hydro-Brake is installed, a 3 in. orifice, constructed from 0.75 in. plywood, was installed at the tank outlet.

Figure 40 presents an overall plan and profile of the off-line storage facility.

Installation of Monitoring Equipment--

To accurately define the areas of surface flooding and sections of sewers experiencing surcharge conditions prior to installing the off-line storage facility and the flow restrictors or Hydro-Brakes at selected catchbasins, it was necessary to adequately monitor sewer flows in the study area.

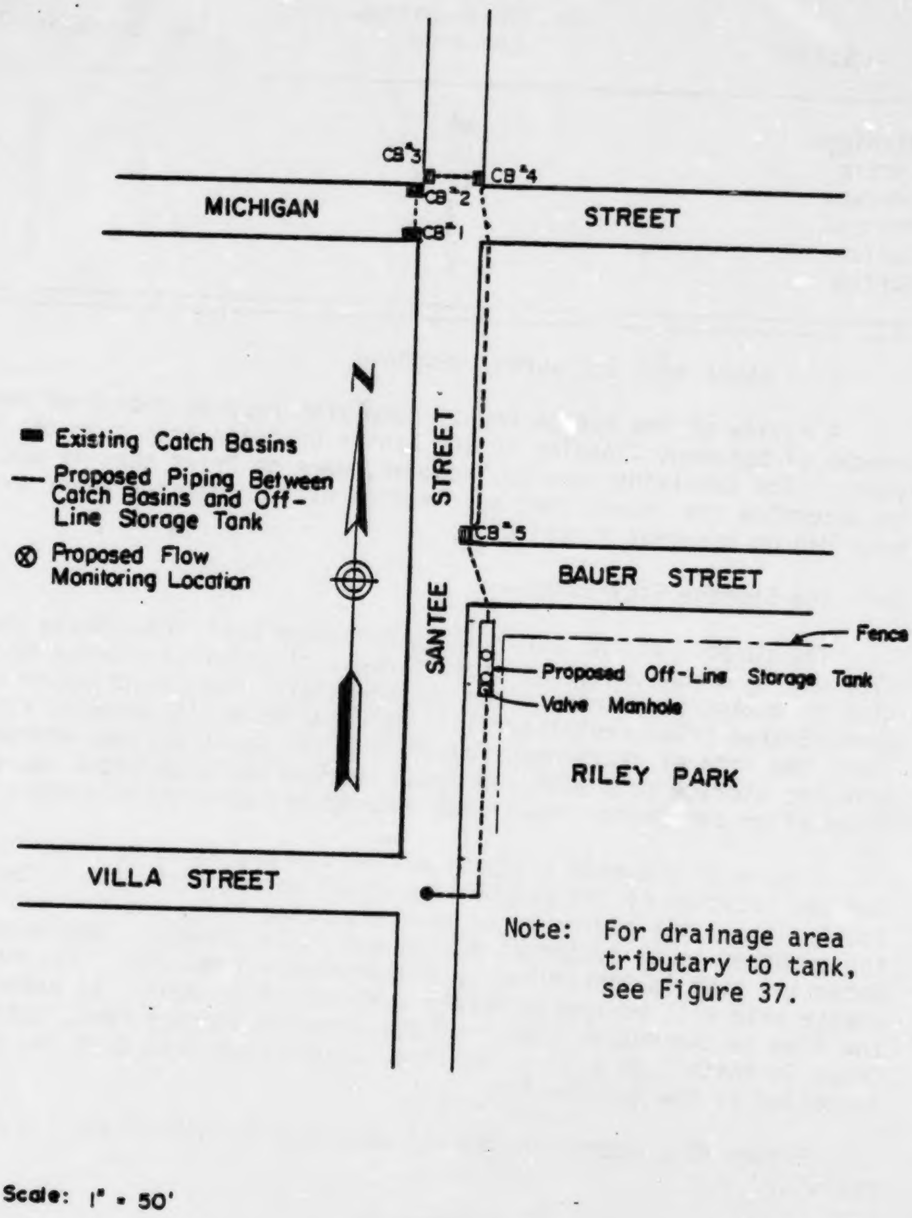

CB#3
CB#4
CB#2
CB#1
MICHIGAN
STREET
N
■ Existing Catch Basins
— Proposed Piping Between Catch Basins and Off-Line Storage Tank
⊗ Proposed Flow Monitoring Location
SANTEE STREET
CB#5
BAUER STREET
Fence
Proposed Off-Line Storage Tank
Valve Manhole
RILEY PARK
VILLA STREET
Note: For drainage area tributary to tank, see Figure 37.
Scale: 1" = 50'

Figure 39. Schematic of off-line storage facilities associated with Santee area Hydro-Brake demonstration site.

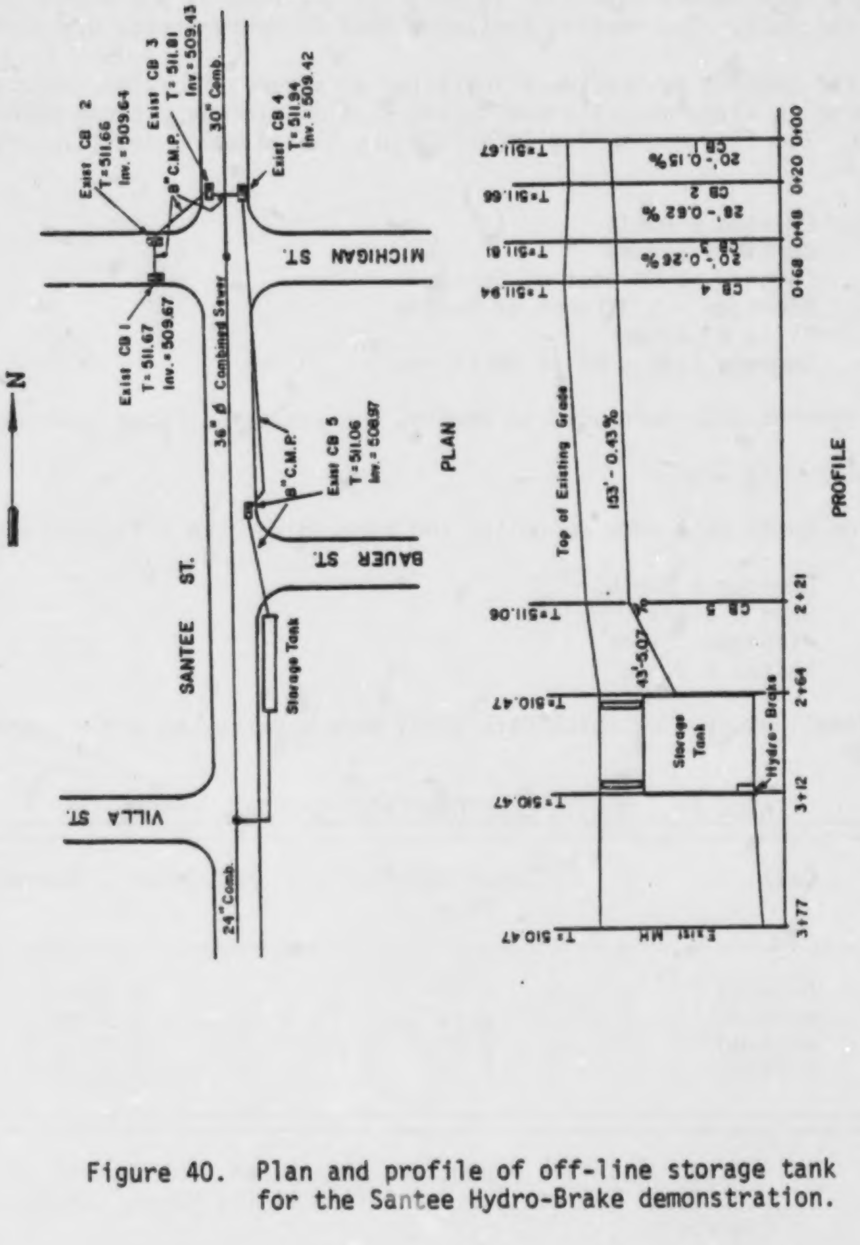

Exist CB 2
T=511.66
Inv.=509.64
Exist CB 3
T=511.81
Inv.=509.43
8" C.M.P.
30" Comb.
Exist CB 4
T=511.94
Inv.=509.42
Exist CB 1
T=511.67
Inv.=509.67
MICHIGAN ST.
36" Ø Combined Sewer
N
Exist CB 5
T=511.06
Inv.=508.97
8" C.M.P.
PLAN
BAUER ST.
SANTEE ST.
Storage Tank
VILLA ST.
24" Comb.
T=511.67
T=511.66
T=511.81
T=511.94
CB 1
20'-0.15%
CB 2
28'-0.62%
CB 3
20'-0.26%
CB 4
0+00
0+20
0+48
0+68
Top of Existing Grade
153'-0.43%
PROFILE
T=511.06
CB 5
2+21
43'-3.07%
T=510.47
Storage Tank
Hydro-Brake
2+64
3+12
T=510.47
Exist MH
T=510.47
3+77

Figure 40. Plan and profile of off-line storage tank for the Santee Hydro-Brake demonstration.

Preliminary stormwater modeling of the sewer system within the Santee drainage area had indicated probable sewer surcharging along Emerson Street. Monitored data collected to date indicated that several sewer sections along this street experience frequent surcharging. Additional modeling studies were conducted to determine the number and location of the catchbasins that will receive a Hydro-Brake regulator to restrict the rate of stormwater inflow during rainfall. The results indicated that 13 Hydro-Brakes are needed.

Level sensing devices were installed at several locations within the study area to allow accurate monitoring of flows throughout the demonstration project. The flow monitoring locations are listed below and illustrated in Figure 41.

1. Emerson & Robin
2. Curtis & Santee
3. Curtis - 1 MH west of Santee
4. Michigan - 1 MH west of Santee
5. Villa & Santee
6. Storage Tank - MH at north end

The monitoring data were used to develop and evaluate system hydrographs.

Flow Monitoring Data--

Flow depth data were collected and reduced for the following sites:

1. Emerson & Robin
2. Curtis Street
3. Michigan Street
4. Santee & Villa

The rainfall events for which data plots were constructed are presented in Table 34.

TABLE 34. MONITORED STORM EVENT RAINFALL PARAMETERS

Date	Total Rain* in.	Max. Hourly Intensity* in.
5-18-80	0.40	0.12/hr
6-06-80	2.19	1.97/hr
6-07-80	2.17	1.05/hr
7-22-80	1.03	0.45/hr

* Note: Rainfall parameters were taken from records at the National Weather Service Office at the Monroe County Airport.

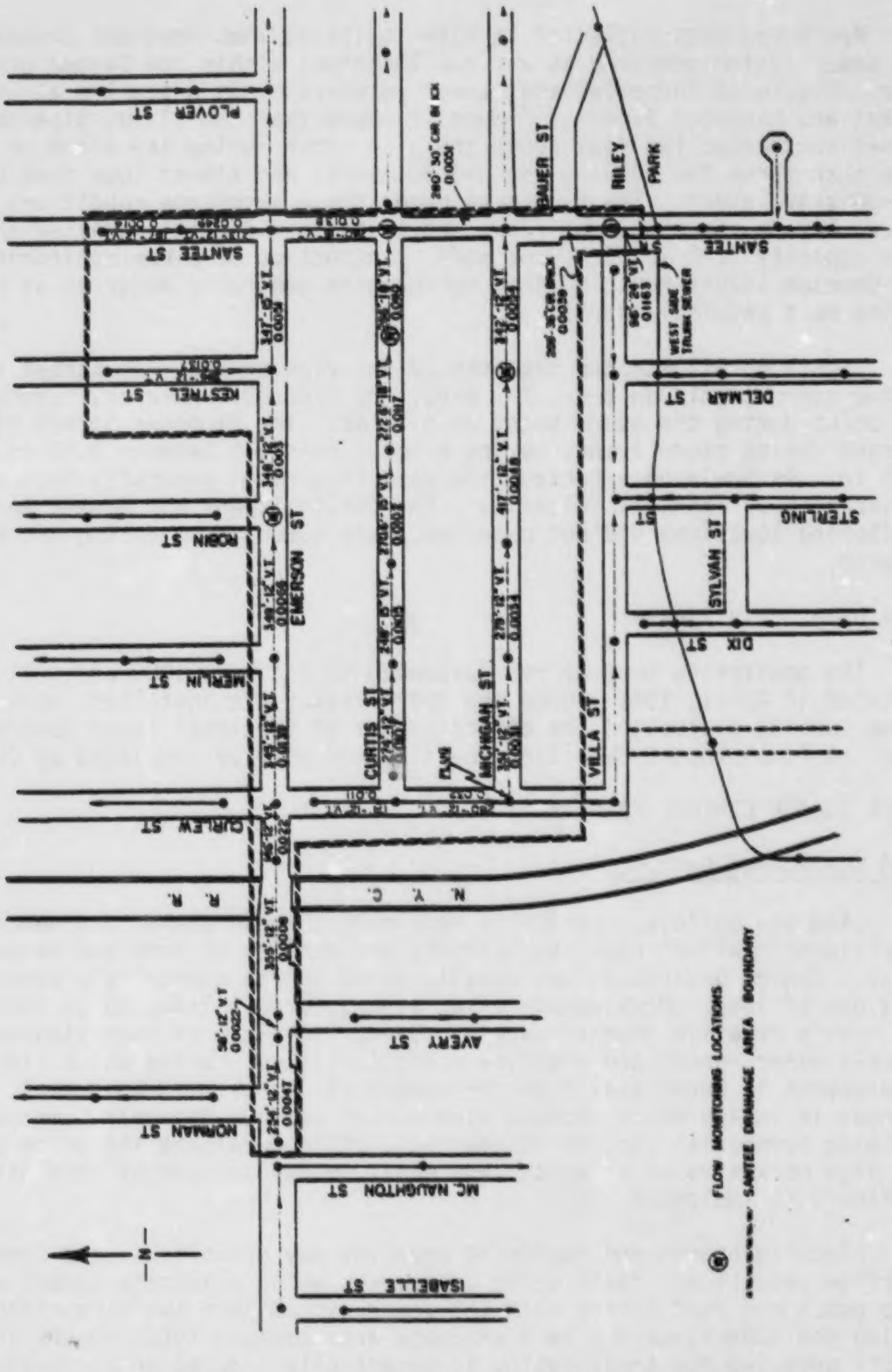

PLOVER ST
SANTEE ST
BAUER ST
RILEY PARK
SANTEE
KESTREL ST
ROBIN ST
EMERSON ST
MERLIN ST
CURLEW ST
CURTIS ST
MICHIGAN ST
VILLA ST
WEST SIDE TRUNK SEWER
DELMAR ST
STERLING
SYLVAN ST
DIX ST
R. R.
N. Y. C. R. R.
AVERY ST
NORMAN ST
MC NAUGHTON ST
ISABELLE ST
N
FLOW MONITORING LOCATIONS
SANTEE DRAINAGE AREA BOUNDARY

Figure 41. Flow monitoring locations within Santee area Hydro-Brake demonstration site.

Monitored data collected to date indicated that frequent surcharging of the sewer system occurred at various locations within the Santee drainage area. Figure 42 indicates that sewer surcharging is a problem at Emerson Street and Michigan Street. Figure 42 shows that the 12-in. pipe on Emerson Street surcharged two feet above the pipe crown during the storm on 5-18-80, more than three feet during the 6-6-80 event, and almost four feet during the 6-7-80 storm event. The flowrates under these surcharge conditions were unknown, but in any case, the flows had to be significantly greater than the pipe capacity of 0.80 cfs (0.52 mgd). Inspection of other monitoring data for Emerson Street revealed that surcharging generally occurred at this site during most rainfall events.

Figure 43 illustrates that the 12-in. pipe on Michigan Street surcharged during storm events on 6-6-, 7-, 8-80, and 7-22-80. However, surcharging did not occur during the storm event on 5-18-80. The Michigan Street site surcharged during storm events having a total rainfall between 0.50 in. and 1.00 in. As would be expected, the peak flow rates generally increased with an increase in rainfall intensity. The Curtis Street and Santee and Villa monitoring locations did not experience any surcharging during the monitoring program.

Remaining Activities--

The monitoring program was suspended in October, 1980 and will be reinstituted in April, 1981. Once the Hydro-Brakes are installed, work will continue towards evaluating the effectiveness of the total inlet control program. A final report detailing the findings will be completed by July, 1981.

OTHER SOURCE CONTROL MEASURES

Land Use/Zoning Restrictions

Land use policies, generally developed through zoning ordinances, can significantly affect both the quantity and quality of combined sewer overflows. Zoning ordinances are usually based on the community's concept of the best use of land. Development of an area generally leads to an increase in the area's relative imperviousness. Since increases in imperviousness adversely alter runoff and overflow characteristics, zoning which limits development is beneficial from the aspect of stormwater management. However, changes in zoning which promote open spaces or less impervious areas (e.g. rezoning commercial land to residential) often devaluate the price of land. The high market value of urban land makes zoning changes of this nature difficult to implement (1).

Planning boards and reviewing agencies may establish codes benefitting overflow reduction. Restrictive ordinances which eliminate direct entry of sump pumps and roof drains into the sewer system have the same effect as reducing the imperviousness in a drainage area because total runoff and peak runoff entering the sewer system is necessarily reduced or attenuated. Specifying the use of porous pavement in those areas with suitable subsoil conditions is another method also having the effect of decreasing imperviousness. Planned open space within developed areas which can be used for both

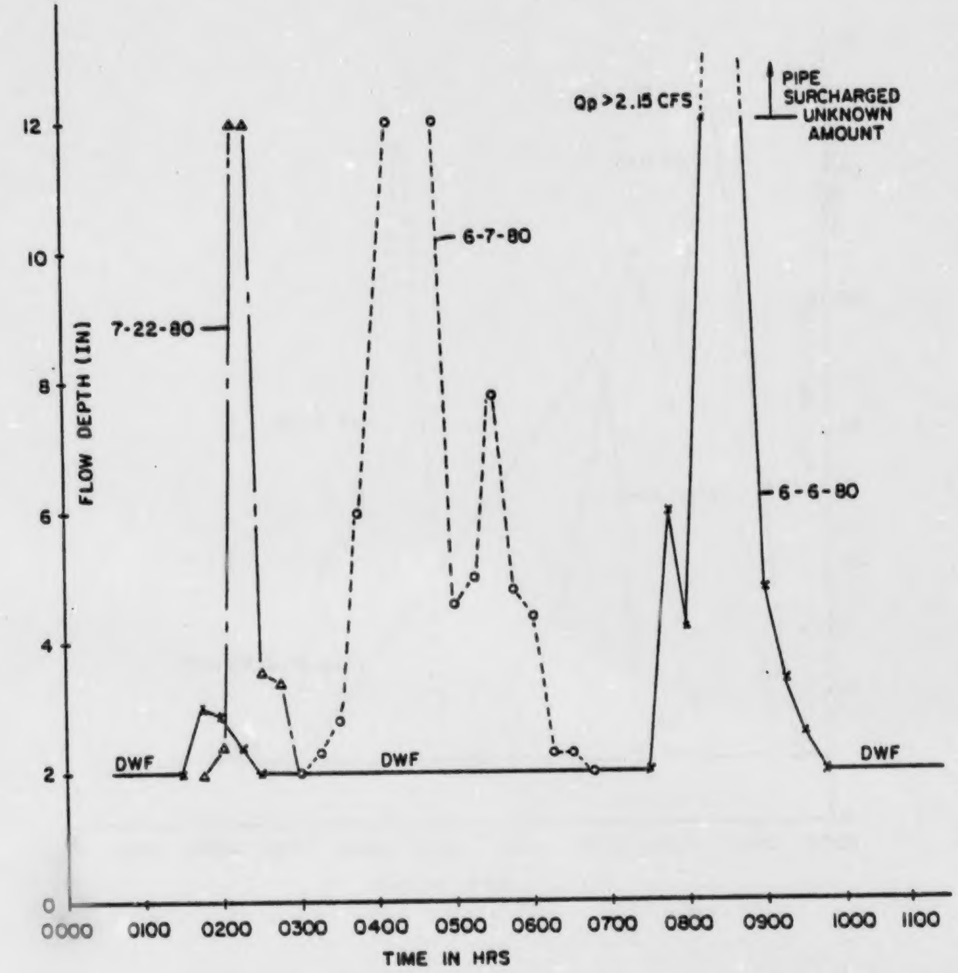


Figure 42. Combined sewage flow depths at Emerson and Robin for selected storms.

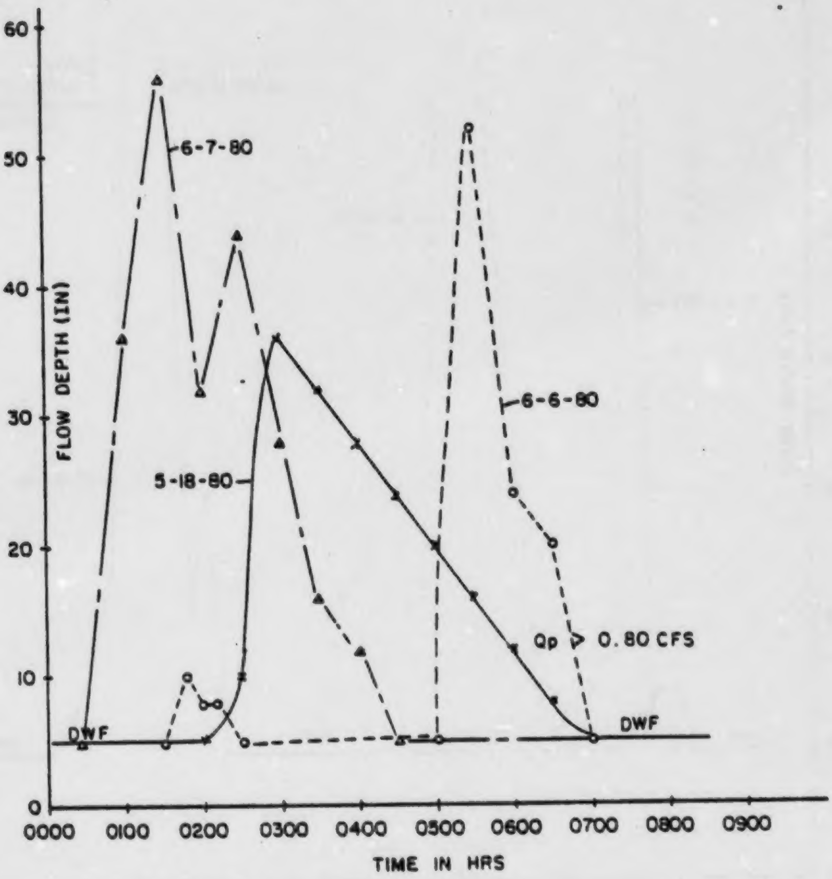


Figure 43. Combined sewage flow depths on Michigan Street for selected storms.

recreation and surface retention is multifunctional and maximizes land use.

An option available to planning boards and reviewing agencies is to require that each land use be associated with a certain percentage of open green areas and/or certain relative imperviousness. Legislating green area development is easier than establishing specific relative perviousness factors because the former has a visible impact on the public's aesthetic awareness. The latter, because of technological consideration, is more difficult to legislate. Therefore, building codes should be established or altered to act as guidelines for attaining relative imperviousness values.

Erosion Controls

A particularly important source of suspended solids in urban runoff is soil erosion from poorly managed construction sites or improperly landscaped land. Erosion can be controlled using such techniques as maintenance of natural vegetation, use of mulches, drainage channel protection modifications, and use of fences to protect trees. Other methods include proper selection of building and highway sites, careful backfilling of pipelines, protection of stockpiled earth, installation of sediment retention basins/dams, and careful clearing and grading during dry seasons.

Although the U.S. Soil Conservation Service (SCS) has no jurisdiction over the operation of construction sites, they do publish guidelines on erosion control that have proved very useful to general contractors. The SCS also provides technical on-site assistance upon request.

The New York State Department of Environmental Conservation (NYSDEC) does require that sediment eroded and washed into rivers and streams be controlled from the standpoint of the detrimental effect on fish and wildlife. In many instances, the NYSDEC may require that various erosion control practices be implemented to limit the discharge of sediment from construction sites.

Surface Retention

Surface retention is a very useful tool in relieving runoff surges that would surcharge storm and/or combined sewers. Surface retention of runoff on a localized basis can be incorporated using various types of detention facilities.

Implementation of surface retention measures is most generally accomplished through local governmental ordinances or building codes. Requirements can be established in newly developing areas for reducing the stormwater runoff rate by specification of particular facilities or by specification of desired performance levels. For example, local ordinances may require that stormwater runoff hydrographs are not to exceed pre-development or natural conditions for rain events below a selected intensity. These provisions are generally applicable to new developments over specified acreages. Monroe County has such an ordinance.

In areas where such ordinances have been adopted, the regulations have served to encourage preservation of open space and utilization of natural drainage concepts, including swales, small detention ponds and rainfall detention on low use open areas (such as playgrounds, parks and parking lots).

In the more intensively developed areas of the City of Rochester, there are obvious incompatibilities with many of these techniques. High land values tend to prevent the preservation of much open space and the demands of heavy pedestrian traffic prevent intentional temporary ponding in areas that might otherwise be suitable. However, a number of more specialized techniques have been developed for intensely developed areas, such as use of intentional rooftop storage and porous pavement.

Buildings with flat-sloped rooftops are used in some places for stormwater detention. Because of rooftop elevations storage can be provided that is not inconvenient for pedestrians or motorists, that is not unsightly and that does not pose a hazard to children. Rooftop storage is not problem-free, however, as there are possible problems from leakage, structural overload and additional maintenance to remove debris and prevent drain blockages which could lead to overtopping (27).

Storing the five year storm on roof tops would amount to an increased stress of about 9 lb/ft^2, at a depth of 1.73 in., which is well below the 40 lb/ft^2 design loads in this region. Thus structural considerations should not present any cost differential relative to conventional building practices. Conventional roofing materials are generally not designed to withstand leakage from pounded water; however, new roofing systems incorporating continuous impermeable membranes are now being marketed at costs competitive with conventional products. Past experience in other locales has indicated that the cost differential between a detention roof and a conventional roof would not be significant if incorporated in new construction or major rehabilitation (28).

The most extensive use of this concept has occurred in the 300 acre Skyline Urban Renewal Project in downtown Denver (28). Since its initiation in 1970, the Denver Urban Renewal Authority has required private developers to temporarily store (on-site) stormwaters directly falling on their properties. In general this requirement is met through the use of rooftop ponding, ponding in plazas and ponding on open grassed areas. The Director of Engineering for the Renewal Authority reports that costs have not proved a problem to developers and that the city's experience with on-site detention has been quite favorable (28). Preparations are now being made to extend the Renewal Authority's development requirments to other areas of the city.

Open areas and park land comprise about 20% of the total surface area in Rochester. Flooding park land and open spaces appears to be more feasible than rooftop storage with respect to total runoff capture, but the disadvantage to flooding is post storm grit and debris clean-up. Because of additional operation and maintenance costs, flooding parkland does not appear to be institutionally feasible. Storage ponds are required for depression storage and therefore this application is significantly limited in developed areas. Surface storage in the urban areas of Rochester would be difficult to implement and therefore is not considered practical.

COST AND BENEFIT CONSIDERATIONS OF SOURCE CONTROL MEASURES

In highly developed urban areas the high price of commercial land generally hinders the implementation of zoning ordinances which promote areas of low imperviousness. Since relatively large amounts of land are required for effective surface retention or detention structures, these control measures are, therefore, practically unfeasible in highly developed urban areas. Rooftop storage, however, appears to be a measure that can be easily implemented in urban areas. Although the increased loads due to rooftop ponding should present no structural problems, special considerations should be given to the watertightness of the roof membrane. In new construction or major rehabilitation, the cost differential between a detention roof and a conventional roof do not appear to be significant (28).

These control measures offer the most promise in newly developed areas where zoning regulations could be implemented which require areas with overall lower imperviousness. This could be accomplished using natural drainage concepts such as swales, ground cover, and natural surface grading. Zoning restrictions could prohibit connection of roof leaders to the sewer system and require planning for surface detention ponds, rooftop storage and porous pavement to decrease imperviousness and reduce stormwater runoff.

The source control management options that appear to be most cost-effective in reducing the pollutants being discharged to all receiving waters from CSO and stormwater discharges are:

- Sewer cleaning and flushing
- Catchbasin cleaning
- Porous pavement
- Inlet control measures
- Erosion control practices.

It is unlikely that any of the above options, by themselves, can totally solve the problem, especially in an urban area. The combination of these identified measures can significantly reduce the pollutant loadings presently occurring to the receiving waters.

It is important to recognize that these measures should be selectively applied throughout the overall drainage basin. A rationale for their implementation should be developed after the specific problems have been identified. That is, for example, sewer cleaning and flushing should be conducted in those areas having the sewers with the most accumulated debris and deposits as determined from field inspections.

Relative to catchbasin cleaning, the exact relationship between the reduction in pollutant discharge and the condition of the catchbasins has yet to be determined. Clogged catchbasins will prevent entry of stormwater into the sewer system which may lessen the probability of CSO, but only at the expense of additional surface flooding. The question then becomes, how much surface storage or flooding is acceptable. This is precisely the question that must be answered before the implementation of inlet control devices such as the Hydro-Brake unit. There are also problems with accumulated solids in catchbasins that may be scoured during heavy rainfalls and discharged into the sewer system. These solids may then become a portion of any CSO discharged from the system.

Given the approximate equal costs for conventional asphaltic pavements and porous pavements, consideration should be given to the use of porous pavements in all new parking areas and when resurfacing existing lots. The general applicability of the use of porous pavements for roadways, especially when considered over a long period of time, has not yet been determined.

A cost-effective inlet control measure are those devices that limit or throttle the rate of inflow into the combined sewer system. Devices such as the Hydro-Brake unit are easily installed in existing catchbasins at very low costs. They are definitely effective in reducing CSO and basement backups but only at the expense of greater surface flooding. For those areas where additional street ponding can be tolerated, then inlet control measures can be a very attractive alternative. Again, the general applicability must be carried out on a site-specific basis.

In general, most of the erosion control measures suggested by authorities such as the SCS, are easily implemented at low cost. These measures should always be considered during any major construction activity.

Although largely dependent on the extent to which the identified source management control options are implemented, it is estimated that selective implementation of various source control measures would result in an annual CSO reduction of 5 - 10%. This assumes that inlet control devices are not used. Obviously, the more stormwater that is throttled at the catchbasins, the greater the decrease in CSO, but at the expense of widespread surface ponding and flooding.

The estimated percent decrease is relatively low and would be so for most urban areas. Source control management options would be more effective in rural and developing residential areas. In these areas, the best option can be constructed along with the planned structural and land development. The needed space requirements are generally available in such areas.

SECTION 7

COLLECTION SYSTEM MANAGEMENT

MINIMAL STRUCTURAL IMPROVEMENTS

Objective

As discussed in the previous section on source control management measures, the expected decrease in annual CSO volume is relatively small for such identified options. The only exception would be the widespread use of inlet control devices. Such widespread use, however, poses other problems that must then be addressed and accepted by the public. For the Rochester BMP program, it was found that several minimal structural improvements to the existing sewer system could result in significant reductions in CSO volume and frequency.

It should be noted that the term, minimal structural, is relative and depends on form whose viewpoint it is considered and the size of the municipality involved. What is meant by the term in this report are those improvements that can be made to the existing sewer system for relatively small expenditures of capital and manpower and that involve a minimal of construction time and effort. The alternative to the minimal structural approach or the BMP concept would be the structural approach. This would involve large capital expenditures and would generally include control measures such as storage and conveyance tunnels and wet-weather treatment facilities. For a large urban area expenditures in the hundreds of millions are typical for the structural approach, as opposed to about 10% of that figure for the minimal structural measures.

It is essential to recognize that the improvements identified as minimal structural may not be sufficient to satisfy the receiving water standards or all of the community's needs in the way of flooding and basement backup relief. A partial structural approach may still be necessary. Moreover, the projected decreases in CSO volume and frequency as discussed in the subsequent sections for the minimal structural approach are based on average annual rainfall conditions taken over a year's time. CSO reductions for specific large storm events are not that significant for the BMP and minimal structural improvements. They would be for the structural alternatives.

At the heart of all of the identified minimal structural improvements is the objective of maximizing the use of the existing sewer collection and treatment system. It seems obvious that the existing system should and can be optimized prior to implementing any structural improvements.

The minimal structural improvements discussed in this section include:

- Selective structural improvements to the interceptor
- Minor modifications and adjustments of most of the overflow regulators
- Increasing the height of the existing overflow weirs by the addition of wood timbers
- Installing a Hydro-Brake regulator (flow controller) at the Lexington Ave. regulating chamber
- Operating the existing VanLare Treatment Plant under a split-flow mode of operation whereby plant efficiency during storm events is increased.

Although the above identified minimal structural improvements are discussed separately herein, these control measures are complimentary in their effectiveness in reducing the frequency and volume of CSO to the Genesee River. Each individual element - interceptor improvements, regulator modifications, weir height changes, and split-flow mode of VanLare STP operation - involved maximizing the use of the existing sewer conveyance and treatment systems. Although reduction in CSO's would occur upon implementation of selected improvements, since these elements are compatible, maximum CSO reductions occur when all the identified alternatives are implemented.

Interceptor Improvements

Background--

Before the construction of the St. Paul Boulevard Interceptor (SPBI) (circa 1912), all of the then existing outlet sewers discharged directly into the Genesee River at various locations north of the Upper Falls. These outlet sewers, now termed trunk sewers, conveyed all wastewater flow from the upstream tributary service areas to the River for disposal. It was evident that with the rapidly growing population of the City of Rochester and surrounding areas, in conjunction with the low flow in the Genesee River during the dry season of about 300 cfs (194 mgd), untreated sewage could not be discharged to the Genesee River on a continuous basis.

To divert sewage entering the Genesee River and convey the wastewater to a treatment plant, an intercepting sewer was constructed to intercept the existing outlet sewers near their discharge outlets. The interceptor naturally followed the general course of the River. Construction of an intercepting sewer in conjunction with treatment provided by a disposal plant located near Lake Ontario was the first major step in pollution abatement for the City of Rochester.

Today, the SPBI diverts sewage in the trunk sewers from the River and conveys it to the Frank E. VanLare Treatment Plant near Lake Ontario. Originally, all of the dry-weather flow in the trunk sewers and two and one-half additional equal volumes of stormwater runoff generated during rainfall were collected. To control the rate of wastewater diverted into the interceptor from the trunk sewers, regulators were constructed at the end of each trunk sewer. Excessive inflow to the interceptor would result in unacceptable surcharge conditions within the SPBI. These regulating devices consisted of float-activated gates which controled the rate of wastewater transfer to the SPBI. A sensing line between the float chamber and the trunk sewer measured the depth of flow in the trunk sewer. Based on the adjustment of the regulator float mechanism, changes in depth of flow in the trunk sewer activated the float which, in turn, opened or closed the gate on the transfer line from the trunk sewer to the interceptor.

The SPBI starts on the east side of the Genesee River immediately above the Upper Falls at the intersection of Central Avenue and Water Street. The reader is referred to Figure 44 for the location of the SPBI. From this point the interceptor runs westerly under the River along Central Avenue to Front Street. Trunk sewers discharging at Central Ave. and Water Street and at Front Street are intercepted. The tunnel between these two locations is approximately octagonal in shape; however, the flow is confined to a semicircular cunette 3 ft wide.

Beyond Front Street the tunnel is approximately 6 ft in diameter, constructed in hard rock and lined with a brick invert. Where soft rock was encountered, the entire circumference was lined with brick. The next interception of a trunk sewer is at Mill and Factory Streets. The former Genesee Valley Canal and Platt Street sewers combine at this regulator chamber. Proceeding downstream the next point of interception is at Spencer Street. The SPBI then proceeds to the siphon entrance at the Cliff Street Screenhouse. From this point to Avenue B on the east side of the Genesee River the flow in the SPBI is carried by two cast iron pipes acting as inverted siphons. A 24 in. pipe is for dry-weather flow and a 42-in. is for additional flow generated during periods of rainfall.

At Avenue B flow from the trunk sewer serving the area known as Carthage is diverted into the interceptor. At this location flow from the west side of the City is conveyed by the West Side Trunk Sewer Diversion (WSTSD) siphons into the interceptor. The flow was originally conveyed by 3 siphons (16, 24 and 30 in. diameter) originating at the Glenwood Avenue Screenhouse. BMP investigations revealed that only the 16 and 30 in. siphons were operating. The 24 in. siphon was completely clogged. Recent cleaning operations removed the accumulated material and all three siphons are presently operating.

From Avenue B, the SPBI continues in a northerly direction being tunneled in rock with a brick lining. Approximately 200 ft south of the intersection of St. Paul Boulevard and Norton Street the tunnel ends and the intercepting sewer is continued in a conduit constructed by open-trench methods. At the above intersection, flow from the East Side Trunk Sewer (ESTS) is taken into the SPBI. A regulator chamber located at Norton and Jewel Streets controls the amount of flow from the ESTS into the interceptor. In recent years a parallel conduit, constructed by open-trench was constructed between the junction chamber at St. Paul Blvd. and Norton Street and the junction chamber at Ridge Road.

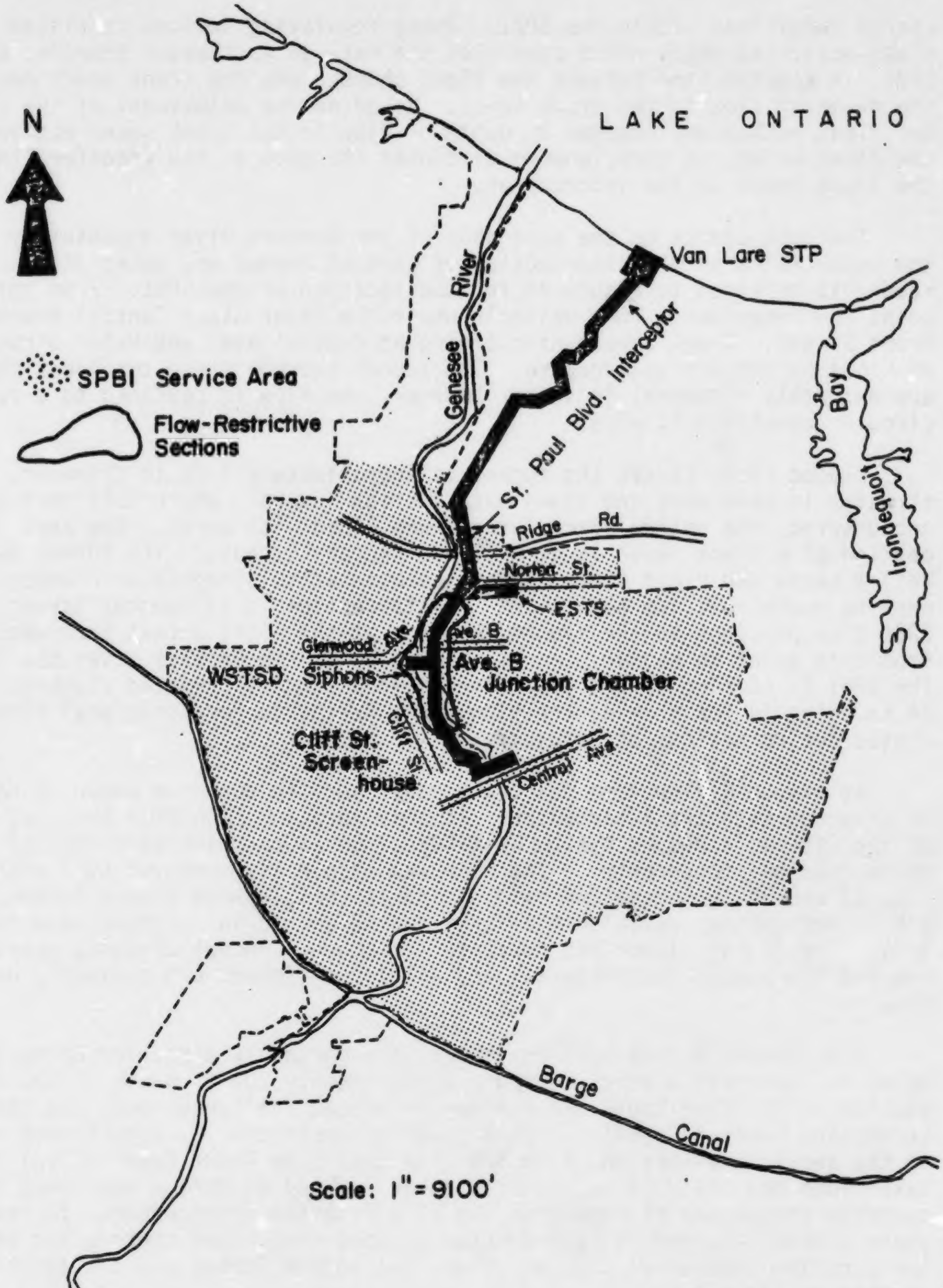

LAKE ONTARIO
N
Van Lare STP
St. Paul Blvd. Interceptor
Genesee River
SPBI Service Area
Flow-Restrictive Sections
Irondequoit Bay
Ridge Rd.
Norton St.
ESTS
Glenwood Ave.
Ave. B
WSTSD
Siphons
Ave. B Junction Chamber
Cliff St. Screen-house
Cliff St.
Central Ave.
Barge Canal
Scale: 1" = 9100'

Figure 44. Location of flow-restrictive sections of the SPBI.

Grit chambers just prior to the siphons under the River at Cliff Street were removed in the 1960's. The chambers in the structure now discharge directly into the siphons. At the end of the Screenhouse channels are overflow weirs discharging into a 24-in. overflow pipe which empties into the River. This overflow pipe was intended for emergencies only, such as when the siphons become clogged and are not able to carry normal flow. Thus, this overflow line rarely receives flow, even during intense rainfall events.

The portion of the SPBI from Ridge Road to the treatment facility was originally designated as the Outfall Sewer. The Outfall Sewer was constructed in such a manner that, regardless of conduit size, uniform carrying capacity from Ridge Road to the treatment plant was maintained. At the time of construction no additional flow entered the SPBI north of Ridge Road. It was anticipated, however, that at some later date the SPBI was to receive sewage from the area north of the city boundaries within the Town of Irondequoit. Figure 44 shows the general alignment of the SPBI from its start to the VanLare Treatment Plant. Also shown are the major trunk sewers discharging into the interceptor at the regulating chambers.

The frequency and volume of CSO presently discharged to the Genesee River are largely dependent on the wastewater transfer rates established at each regulating structure. Flowrates within the interceptor are established by inflow from the trunk sewers as controlled by the regulating devices. The regulators were initially adjusted such that the conveyance capacities throughout the interceptor would not be exceeded, that is, the SPBI was designed to flow under open-channel flow conditions and not become surcharged. Figure 45 shows the full-flow conveyance capacity of the SPBI along its entire route.

To reduce the frequency and volume of CSO presently discharged to the River, the regulators could possibly be adjusted to allow for greater wastewater transfer rates than those previously established. As seen from Figure 45, however, there are several sections along the SPBI which have conveyance capacities inconsistent with adjacent sections. Those sections are identified as follows and are shown in Figure 44:

- The section of the interceptor from the Avenue B junction chamber to the intersection of Norton Street and St. Paul Boulevard
- The interceptor siphons under the Genesee River from the Cliff Street Screenhouse to the Avenue B junction chamber
- The siphons under the Genesee River from the Glenwood Screenhouse to the Avenue B junction chamber.

The siphons from the Glenwood Screenhouse to the SPBI are included as flow-restrictive because of their importance in sewering the west side of the City of Rochester. Almost the entire area within the City west of the Genesee River is served by a combined sewer system with the wastewater being transferred to the interceptor by siphons originating at the Glenwood Screenhouse.

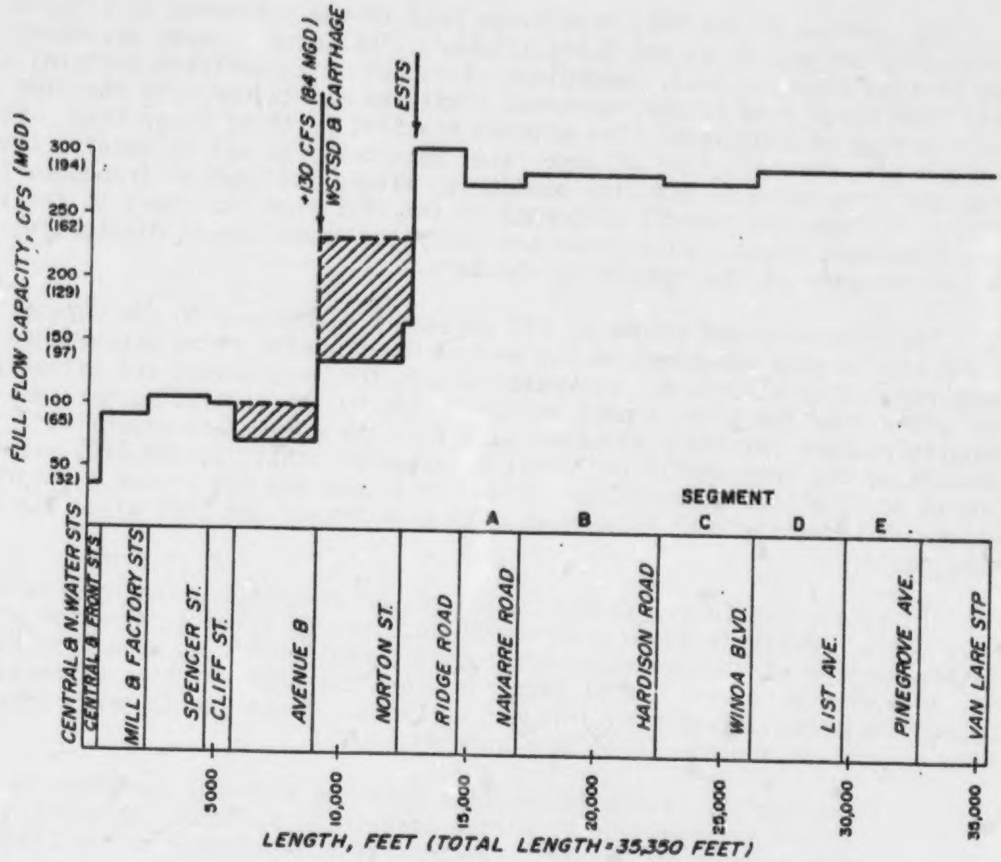


Figure 45. Hydraulic capacity profile of the St. Paul Boulevard Interceptor.

Any improvements or adjustments to the overflow regulators in an effort to reduce CSO discharges, if not accompanied by removal of these flow-restrictive sections of the SPBI, will not result in optimized CSO reductions. Overloading the interceptor, by allowing excessive inflow rates at the regulators, would eventually lead to overflow relief at the Cliff St. Screenhouse.

Field Survey--

To properly assess the full capabilities of the SPBI in terms of accepting more flow from the trunk sewers, the present condition of the interceptor was determined. The previous evaluation of the SPBI was conducted in 1969 as part of a comprehensive sewerage study for the City of Rochester (6). At that time, it was noted that the portion of the brick sections of the interceptor were in good condition. There was no evidence of need for rehabilitation of the sewer except for replacement of manhole steps. Within the tunnel portions of the SPBI, the field inspections revealed that the sewer was relatively free of rock or other debris that might adversely affect its carrying capacity.

Under the BMP program, an intensive field survey was conducted to determine the present status of the intercepting sewer. Of primary importance as related to subsequent modeling of the system and hydraulic analysis to evaluate the flow-restrictive segments, was verification of sewer sizes, manhole inverts, and structural condition. An attempt was made to determine the invert and rim elevations for all manholes along the SPBI, the size and shape of the conduit or tunnel sections, and an estimate of the roughness coefficient to use in computing flowrates and conveyance capacities.

The field inspection indicated that, in general, the SPBI from its origin to the VanLare Treatment Plant, is in structurally good condition. Field measurements were very close to values taken from the original design plans and other reports. Furthermore, velocity measurements taken by a hand-held velocity meter in conjunction with measured depth of flow readings established an average 'n' value or roughness coefficient of 0.013. This agreed favorably with generally accepted values for brick sewer conduits operating under openchannel flow regimes. Velocity readings obtained by the meter were verified by dye testing between manholes along the interceptor. Table 35 highlights the close comparison between the design and measured field slopes for the portion of the interceptor originally known as the Outfall Sewer. Other portions of the SPBI showed similar comparisons.

TABLE 35. COMPARISON OF DESIGN TO FIELD MEASUREMENTS ALONG THE MAIN INTERCEPTOR

Segment	Slope (ft/ft)	
	Design	Field Measured
A	0.0025	0.00247
B	0.0050	0.00512
C	0.0050	0.00505
D	0.0033	0.00300
E	0.0100	0.00976

Note: Refer to Figure 45 for segment locations.

Rationale for Selective Interceptor Improvements--

A review of the overall interceptor and trunk sewer system was conducted to establish a rationale for selective interceptor improvements that would reduce CSO frequency and volume. The following discussion summarizes the considerations in establishing that rationale.

Overflow from portions of the East Side Trunk Sewer (ESTS) can be diverted into the Cross-Irondequoit Tunnel (CIT) by the Culver-Goodman Tunnel system (under construction) prior to discharge to Irondequoit Bay. All wastewater that enters the CIT, however, requires pumping to the treatment plant. Flow within the SPBI, however, is conveyed to the same treatment facility by gravity. To eliminate daily pumping of ESTS flow that could discharge into the CIT dictated keeping all dry-weather flow in the ESTS. Based on estimates of population, infiltration, and industrial flows, the total dry-weather flow (DWF) in the ESTS is approximately 40 cfs (26 mgd).

Additional wet-weather flows taken into the SPBI would decrease the volume of overflow discharged at various system relief points. To allow for greater inflow rates from other trunk sewers, inflow to the interceptor from the ESTS of only dry-weather flow would be necessary. Presently, the regulator located at Norton and Jewel Streets diverts approximately 130 cfs (84 mgd) into the SPBI. Due to future overflow relief from the ESTS to the CIT during rainfall events, overflow elimination from the system while limiting flow in the ESTS to the dry-weather flow of about 40 cfs (26 mgd) would be possible. From Figure 45 it is seen that the capacity of the SPBI downstream from Norton Street to the treatment plant varies between 270 cfs (174 mgd) and 310 cfs (200 mgd). However, the section of the interceptor between the siphon chamber and Norton Street has a capacity of only 150 cfs (97 mgd). Therefore, to make this section of the interceptor hydraulically compatible with the downstream section, and taking into account the ESTS maximum diverted flow of 40 cfs (26 mgd), an additional 90 cfs (58 mgd) of capacity is required.

With the available elevation difference between the Avenue B siphon outlet and the interceptor junction with the ESTS a conduit approximately 5 ft in diameter would be sufficient to convey the additional required 90 cfs (58 mgd). Proceeding upstream, decreasing overflow to the Genesee River from the WSTS would require an increase in capacity from the Glenwood Screenhouse to the SPBI. Presently, the combined capacity of the incoming conduits to the junction chamber located at Avenue 'B' is approximately 135 cfs (87 mgd). This flowrate represents 60 cfs (39 mgd) from the interceptor siphons, 60 cfs (39 mgd) from the WSTS siphons, and 15 cfs (10 mgd) for the sewer serving the Carthage drainage area.

The regulator diverting flow from the Carthage service area into the SPBI can be structurally modified to allow a 35 cfs (23 mgd) diversion into the interceptor. Then, 205 cfs (132 mgd) of conveyance capacity would remain to be utilized to maximize use of the SPBI. The Carthage drainage area is known to be high-impacting in terms of CSO pollutant loading generated during storm events because of one major industry. By accepting more wastewater into the interceptor, the pollutional load from this overflow location will be decreased. Modifying the Carthage regulator to permit a 35 cfs (23 mgd) diversion to the SPBI would involve structural changes in excess of those termed minimal structural. As such, this transfer rate will not be achieved under the BMP program. A Step 1 USEPA grant for such improvements has been applied for and received. In the interim, however, minor adjustments can be made to the Carthage regulator that will allow a partial increase in the transfer rate. This is discussed in more detail in the subsequent section on regulator modifications.

By increasing the WSTS siphons to convey a total 120 cfs (78 mgd) from the present capacity of 60 cfs (39 mgd), overflow could be decreased from the west side system. In addition, the capacity of the interceptor siphons could be increased to 85 cfs (55 mgd) from the present 70 cfs (45 mgd) which could decrease the present overflow quantities from the Mill and Factory relief point. Increasing the WSTS siphons to convey 120 cfs (78 mgd) would involve the installation of a 36-in. diameter conduit, whereas increasing the interceptor siphons by 15 cfs (10 mgd) would require an additional 24-in. diameter conduit.

In summary, to insure that a uniform maximum flow is maintained in the SPBI with subsequent CSO reductions, the modifications as noted in Table 36 would be required. Costs associated with these identified improvements are presented in Section 9.

TABLE 36. SELECTIVE INTERCEPTOR IMPROVEMENTS

Location	Required Length (ft)	Required Size (in.)
Interceptor Siphons	3400	24
Interceptor Avenue B to Norton Street	3400	60
West Side Trunk Sewer Diversion Siphons	800	36

Projected Overflow Reductions--

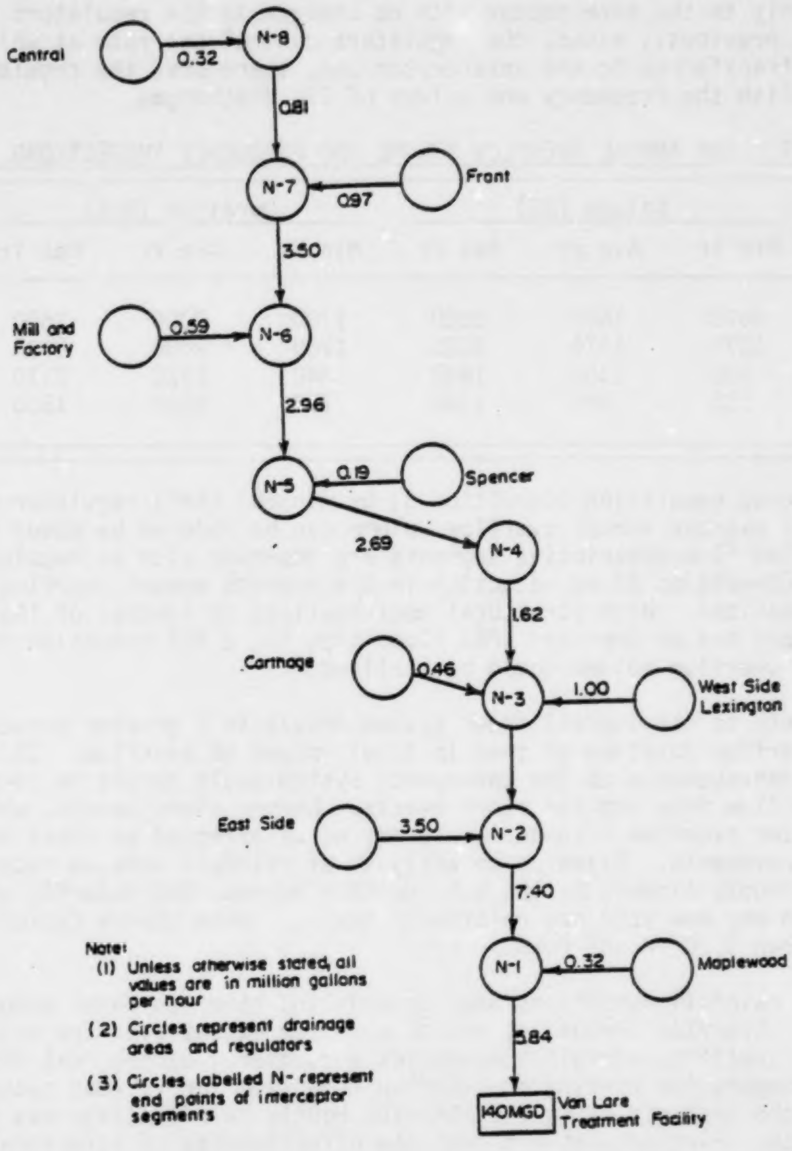
To quantify the frequency and volume of CSO discharged from the present SPBI system with the identified flow-restrictive segments, simplified stormwater modeling simulations were conducted. The Simplified Stormwater Model (SSM) developed by Metcalf & Eddy in conjunction with the Rochester Combined Sewer Overflow Program was used (1).

Figure 46 shows the schematic representation of the existing SPBI that was modeled using the SSM. The modeled regulators included the following:

(1) Maplewood
(2) East Side Trunk
(3) West Side Trunk Sewer and Lexington
(4) Carthage
(5) Spencer
(6) Mill and Factory
(7) Front
(8) Central

From 20 years of hourly rainfall records (1954-1973) three years were selected for model simulation. These years included the maximum, minimum and average years relative to total annual precipitation within the 20 yr record. With this rainfall base four sets of system conditions were evaluated. They were:

1. Existing SPBI with unimproved flow-restrictive segments and unmodified regulators/weirs
2. Removal of flow-restrictive sections of SPBI with no modifications to regulators
3. Existing SPBI with modified regulators as implemented under the BMP program (regulator adjustments and weir height increases)
4. Removal of flow-restrictive sections of SPBI with BMP modified regulators and control structures.


Central
0.32
N-8
0.81
N-7
Front
0.97
3.50
Mill and Factory
0.59
N-6
2.96
N-5
0.19
Spencer
2.69
N-4
1.62
Cartnage
0.46
N-3
1.00
West Side Lexington
East Side
3.50
N-2
7.40
N-1
0.32
Maplewood
5.84
140MGD
Van Lare Treatment Facility

Note:
(1) Unless otherwise stated, all values are in million gallons per hour
(2) Circles represent drainage areas and regulators
(3) Circles labelled N- represent end points of interceptor segments

Figure 46. Schematic of Existing Sewer System for City of Rochester for SSM Analysis.

Table 37 and Figure 47 summarize the model projected reductions in CSO frequency and volume for various options. As indicated, the total volume and duration of overflow are unaffected, relative to existing Condition 1, for improvements only to the interceptor with no changes to the regulators (Condition 2). As previously noted, the regulators control the rate at which wastewater is transferred to the interceptor and, therefore, the regulators actually establish the frequency and volume of CSO discharges.

TABLE 37. SSM ANNUAL OVERFLOW VOLUME AND FREQUENCY PROJECTIONS

	Volume (MG)			Duration (Hrs)		
Condition	Min Yr	Ave yr	Max Yr	Min Yr	Ave Yr	Max Yr
1	1070	1670	2020	1700	2000	2690
2	1070	1670	2020	1700	2000	2690
3	930	1560	1840	1340	1520	2110
4	520	990	1140	750	1080	1330

With improved regulation (Condition 3) by minimal (BMP) regulator modification, total average annual overflow volume can be reduced by about 6.5%. If the identified flow-restrictive segments are improved with no regulator modifications (Condition 2) no reduction in the average annual overflow volume would be realized. With structural modifications to several of the system regulators and an improved SPBI (Condition 4), a 41% reduction in average annual overflow volume could be realized.

Improvements to the overall sewer system result in a greater percentage decrease in overflow duration of than in total volume of overflow. This results because improvements to the conveyance system would result in complete capture of overflow from smaller storm events. Larger storm events, which result in greater overflow volumes, would not be as affected by these minimal system improvements. Based on an analysis of rainfall data as recorded at the Monroe County Airport by the U.S. Weather Bureau, the majority of storm events in any one year are relatively small. These storms typically contain less than 0.20 in. of rain.

The three rainfall conditions used in modeling refer to total annual precipitation. Overflow frequency, and to a lesser extent overflow volume, are related to specific rainfall intensities encountered during real storm events. For comparative ranking of overflow reduction for various system improvements, the analysis using the SSM with hourly rainfall data was considered adequate. Furthermore, although the effectiveness of structural system improvements identified as condition 4 was presented here, more will be discussed in subsequent sections.

From known pollutional characteristics of the various overflow discharges, a greater percentage decrease in pollutant loading would be exhibited than that shown for volume reduction. This results from the ability of the upgraded system to capture more of the "first-flush" of an overflow

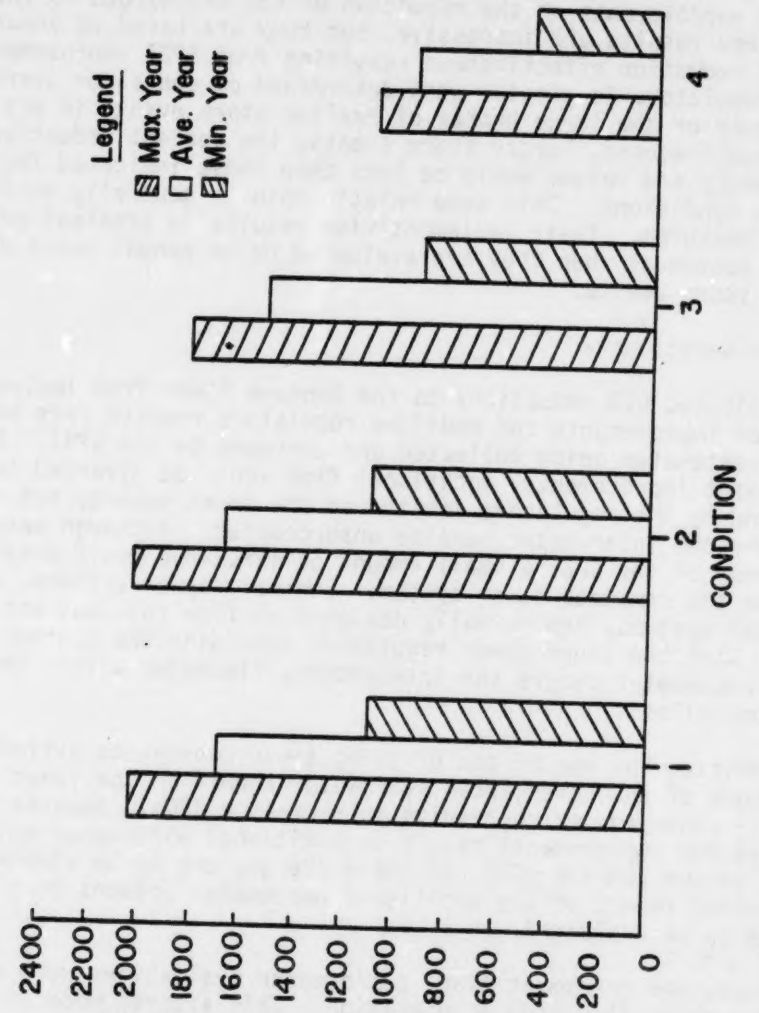


Figure 47. Projected annual overflow volume under various BMP improvement concepts.

event. The highest pollutant concentrations in CSO's generally occur within the first portion of the discharge usually within the first 10 to 30 min. An upgraded sewer system consisting of interceptor improvements and regulator modifications would convey more of the initial portion of increased wastewater flows generated during rainfall events, thereby capturing more of the "first-flush" load.

The modeling projections have been presented in terms of annual rainfall and overflow reductions. Table 37 and Figure 47 illustrate the effects of all system improvements on the reduction of CSO discharged to the Genesee River. These results are impressive, but they are based on annual quantities. CSO reduction effectiveness resulting from SPBI improvements and BMP modified regulators is greater when determined on annual or average conditions because of the large number of smaller storm events in any given year. For the less frequent, larger storm events, the percent reductions in overflow frequency and volume would be less than those indicated for annual precipitation conditions. This same relationship is generally valid for most other BMP measures. Their implementation results in greatest potential for pollution abatement when they are evaluated on an annual basis and for low intensity storm events.

Other Considerations--

Anticipated CSO reductions to the Genesee River from implementation of interceptor improvements and modified regulators results from additional combined wastewater being collected and conveyed by the SPBI. For the identified system improvements, additional flow would be diverted into the interceptor by the regulators located on the trunk sewers, but only to the point where the interceptor remains unsurcharged. Although based on the field survey of the SPBI a small amount of surcharge would present no adverse effects on the combined sewer system. Sanitary sewer systems, as opposed to storm sewer systems, are normally designed to flow full but not become surcharged. With the trunk sewer regulators providing the control over the rate at which wastewater enters the interceptor, flowrates within the SPBI can be easily controlled.

Maximizing the use of the existing sewer conveyance system involves keeping more of the intercepted stormwater runoff in the sewer system that previously conveyed, which results in decreased CSO's. Because of this, the identified BMP improvements result in additional wastewater volumes being conveyed to the VanLare STP. If these changes are to be viable alternatives, the potential impact of the additional wastewater volumes on plant performance had to be evaluated.

Wet-weather treatment plant performance evaluations were conducted which led to the split-flow mode of operation. This altered mode of operation is discussed in detail in subsequent sections. All such performance evaluations indicated that conveyance of additional wastewater flows to the VanLare STP during wet-weather events would have no detrimental effect on plant performance if the plant were operated under the split-flow mode.

A Step 1 USEPA Construction Grant was applied for and received for the identified interceptor improvements herein. Work is presently ongoing (29).

Regulator/Weir Modifications

Background--

As indicated previously, the frequency and volume of CSO presently discharged to the Genesee River are dependent on the wastewater transfer rate established by the regulating structure at the end of each trunk sewer. Selective improvements to the interceptor, if not accompanied by modifications to the regulators to increase this transfer rate, would result in no change in present CSO volume or frequency. This was shown in Table 37 under condition 2.

The subsequent discussion describes the approach taken to selectively modify the existing regulators, which complements the proposed interceptor improvements. Some regulator modifications were actually implemented under the BMP program; whereas, others involving more structurally-intensive improvements, were not but were identified for USEPA Construction Grants funding. In addition, several proposed regulator modifications could not be implemented until after the identified interceptor improvements have been completed.

Field Survey--

As the first step in assessing the overall situation, a detailed field inspection was conducted to determine the present condition and operation of the regulators, as well as the feasibility of modifying the present mode of operation. Upon completion of the inspection a delineation was made between minimal modifications that would be considered a BMP approach and a program requiring more structurally-intensive modifications.

Prior to summarizing the results of the field survey, it is important to describe the configuration and operation of the overflow regulators. Figure 48 is a schematic of a typical regulator/weir overflow chamber. Flow in the trunk sewer is conveyed through the regulator by a steel casting, usually rectangular in shape. Normal dry-weather flow discharges through this casting under openchannel flow conditions. As the flow increases in the trunk sewer, flow through this section occurs under orifice control.

On the downstream end of the casting is a movable orifice plate that can open fully or partially obstruct the opening. Movement of the plate is controlled by a float situated in a chamber adjacent to the plate/pivot arm assembly. A small conduit from the trunk sewer to the float chamber allows the float to monitor the wastewater level in the trunk sewer. Thus, the wastewater level in the trunk sewer controls the operation of the float and, therefore, controls the position of the orifice plate.

In conjunction with the float/orifice plate mechanism, a weir located in the trunk sewer immediately downstream of the steel casting and level sensing conduit allows wastewater to rise to weir elevation before overtopping and

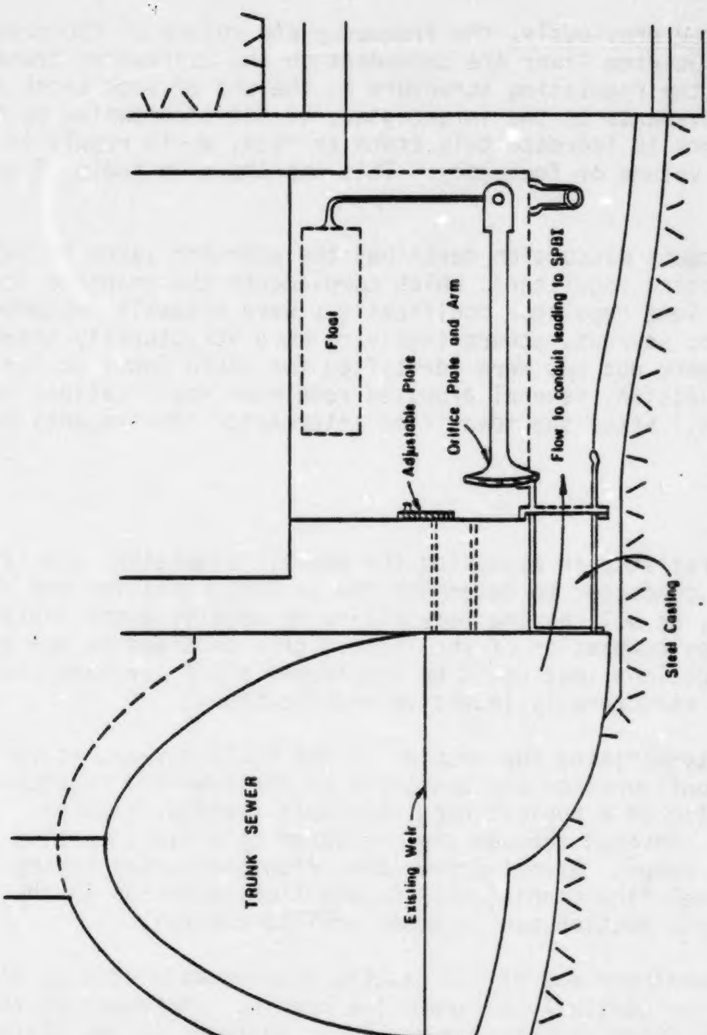

Float
Orifice Plate and Arm
Adjustable Plate
Flow to conduit leading to SPBI
Steel Casting
TRUNK SEWER
Existing Weir

Figure 48. Schematic of typical float-operated regulator.

discharging to the receiving waters. These overflow weirs are either constructed of concrete or consist of wood planks set in the walls of the trunk sewer.

A number of generalized remedial measures were identified as possible modifications to the regulators. These potential measures included:

1. Increasing the casting (orifice) size
2. Replacing the float/orifice plate with an electrically or hydraulically operated sluice gate
3. Adjusting the tilt arm to alter the position of the orifice plate relative to float level
4. Altering the elevation of the overflow weir

Measures 1, 3, and 4 were considered to be practical under the BMP program, while measure 2 was established as structurally-intensive.

Field inspection of the regulators led to the following conclusions:

- Almost all of the regulators operated in such a manner that under storm conditions, the wastewater transfer rate from the trunk sewer to the interceptor decreased. Therefore, to optimize existing system performance and reduce CSO, the present mode of regulator operation had to be altered. This assumed that sufficient conveyance capacities were available in the SPBI.

- Some of the regulators do not function properly because of rust and the corrosive sewer environment.

- Most importantly, however, it appeared that many of the regulators could be modified with minimal effort.

Hydraulic Analysis--

Before the implementation of any regulator/weir modifications, a hydraulic analysis was conducted to determine the actual operating condition of each regulator under varying trunk sewer flow conditions. Some of the parameters that were accounted for in the investigation included:

- Slope, size, and shape of the incoming trunk sewer
- Elevation and size of the steel casting that allows wastewater transfer from the trunk sewer to the interceptor
- Relationship between flow level in the trunk sewer and the corresponding float level and orifice plate positions.
- Length, type, and height of existing overflow weir
- Conveyance capacity of the downstream regulator conduit which transfers wastewater to the interceptor

Table 38 presents a summary of the findings of the hydraulic analyses for the overflow locations with float/orifice controlled regulators.

TABLE 38. OPERATING CHARACTERISTICS OF EXISTING REGULATORS

Name	Site	Design Dry-Weather Diversion Rate (mgd)	Design Wet-Weather Diversion Rate (mgd)	Maximum Potential Diversion Rate (mgd)	Rate Increase Attainable (mgd)
Maplewood	7	8	5	8	3
Lexington	10	5	3	13	10
WSTS	11	22	19	68	48
Carthage	31	10	9	16	7
Mill & Factory	21	11	14	17	3
Spencer	17	5	4	15	11

Note: Maximum diversions were calculated on the basis of maximum orifice and gate openings possible with existing regulator structural configuration. For calculation of design wet-weather and maximum achievable diversion, a pool depth elevation equal to that of the crown of the incoming trunk sewer at the overflow dam was used to establish maximum available head.

Three of the eight monitored overflow locations did not involve float/orifice controlled regulators. At the Front Street location (Site 22), there originally was a float-activated regulator similar to the other structures. Within recent years, however, the float and radial gate were removed and the small steel casting replaced with a much larger opening. In addition to these modifications the height of the overflow weir was substantially increased.

Wastewater transfer from the trunk sewer (Inner Loop Tunnel) at Central (Site 36) is accomplished by conveying flow in an 18-in. diameter conduit which starts immediately upstream of the overflow weir. The only other overflow site which does not involve a regulator is that at the ESTS, known as Seth Green (Site 27). At this location, wastewater transferred from the ESTS to the SPBI is controlled by a manually operated sluice gate installed in a conduit leading from the trunk sewer to the interceptor. A preliminary hydraulic analysis was also conducted for these structures. Table 39 summarizes the findings.

TABLE 39. OPERATING CHARACTERISTICS FOR OVERFLOW SITES WITHOUT REGULATORS

Location	Site	Wet-Weather Maximum Diversion Rate (mgd)
Seth Green	27	84
Front	22	42
Central	36	8

Based on the findings of the hydraulic analyses, Table 40 presents the regulator modifications that were subsequently implemented under the BMP program.

TABLE 40. SUMMARY OF IMPLEMENTED REGULATOR MODIFICATIONS

Location	Site	Modification
Maplewood	7	Fixed radial gate in full open position
Seth Green	27	Closed sluice gate to 50% of full opening
WSTS	11	None[1]
Lexington	10	None[2]
Carthage	31	Fixed radial gate in full open position
Spencer	17	Fixed radial gate at larger opening
Mill & Factory	21	Fixed radial gate at larger opening
Front	22	Structural change required[3]
Central	36	Structural change required[3]

[1] Regulator setting to be adjusted after implementation of identified SPBI improvements.

[2] Float-activated regulator removed-replaced with Hydro-Brake flow controller.

[3] See text.

No minimal changes to the WSTS regulator were scheduled because the actual conveyance restriction was the siphon capacities from the Glenwood Screenhouse to the junction chamber at Avenue B. Until improvements to these siphons are implemented, no additional flow can be transferred to the interceptor from the WSTS.

As part of the overall BMP program a Hydro-Brake regulator was installed and evaluated at one of the system's regulating points. Lexington (Site 10) was selected for such an installation. This is described in detail in subsequent portions of this Section.

As discussed in the section on interceptor improvements, the rate of inflow to the SPBI from the Front Street diversion chamber is presently satisfactory. Structural modifications are necessary, however, to provide for inflow hydraulic control. By controlling the inflow rate, upstream in-system storage within the Main and Front Street tunnels can be realized. Installation of electrically-operated sluice gates, which can provide the needed control, were considered a structurally-intensive control option. Therefore, changes to the Front Street structure were not made under the BMP program. A Step 1 Construction Grant was applied for and received for these structural changes (29). The reader is referred to a later section dealing with structural system modifications.

Based on downstream SPBI capacity and trunk sewer inflow rates, the transfer rate from Central area (via the Inner Loop tunnel) could be increased to 25 cfs (16 mgd). Although they do not represent minimal BMP modifications, several options for increasing the transfer rate include:

- Utilizing the 6 ft diameter unlined rock tunnel instead of the 18-in. pipe and providing a Hydro-Brake flow controller at the upstream end to limit flow to the desired rate.
- Replacing the 18-in. pipe with one of larger diameter.

Since these options represent structurally-intensive options, implementation of either was not made under the BMP program. Additional funding under the USEPA Construction Grants program should provide for these improvements. Preliminary analysis conducted under the BMP program indicated that use of the 6 ft tunnel appeared to be the most viable.

The Carthage combined sewer drainage area has been reduced in recent years because of ongoing urban renewal programs, which also involve sewer separation. Carthage - overflow Site 31 - had been considered high-impacting because of the high ratio of pollutant per unit volume of CSO. Since much of the area is presently served by storm sewers, frequency and volume of CSO should be reduced. The magnitude of this expected reduction could not be determined from the BMP overflow monitoring program.

Sampling of the Carthage overflow, however, revealed that high total pollutant loadings still exist. Therefore, although the wastewater transfer rate should be increased from Carthage, interceptor and regulator hydraulic evaluations indicated that structural modifications would be necessary to optimize use of various conveyance systems. Based on the identified SPBI improvements, a transfer rate of 35 cfs (23 mgd) would be consistent with downstream conveyance capacities and also minimize the high-impacting Carthage CSO. A partial, interim improvement should be made that was consistent with minimal BMP concepts. Removal of the orifice plate/float mechanism would allow for a maximum wastewater diversion rate of about 25 cfs (16 mgd). This represents a 47% increase over the present transfer rate as controlled by the orifice plate/float operating system.

In addition to the regulator modifications, the height of the overflow weirs at most of the diversion structures were increased. This increase allowed for realization of potentially available in-system storage. All scheduled weir height increases were implemented in stages. In this manner, an accurate assessment was made of the effectiveness of each change. It further insured that, if excessive and adverse backwater conditions did develop in the trunk sewers, measures could be quickly taken to relieve such conditions.

The weir height increases were accomplished by installing 6 in. x 6 in. timbers on top of the existing weir base. Angle iron was placed along the side walls of the trunk sewer and spikes and reinforcing bars were placed through the timbers. This resulted in a structurally sound modified weir. Table 41 indicates the sites at which weir increases were implemented and the magnitude of the change.

TABLE 41. SUMMARY OF WEIR MODIFICATIONS

Location	Site	Weir Height Increase - ft
Maplewood	7	2.00
Seth Green	27	None
WSTS	11	0.50
Lexington	10	2.33
Carthage	31	1.33
Spencer	17	1.00
Mill&Factory	21	1.00
Front	22	2.00
Central	36	3.00

As the first step in assessing in-system storage potential, a general review was made of the physical characteristics of the trunk sewers upstream of the overflow regulators. Such factors as sewer size, slope, and length were most important in determining the potentially available in-system storage volumes. The volumes given in Table 42 were based on the increased weir heights (identified in Table 41) and evaluated under static hydraulic conditions in the trunk sewers. The resulting storage volumes are conservative in that the flow dynamics of the system were ignored.

TABLE 42. IN-SYSTEM STORAGE VOLUMES REALIZED BY OVERFLOW WEIR HEIGHT INCREASES

Location	Site	In-System Storage Volume - MG
Maplewood	7	0.05
Seth Green	27	N/A*
WSTS	11	0.43
Lexington	10	0.11
Carthage	31	0.10
Spencer	17	0+
Mill & Factory	21	0.12
Front	22	0.16
Central	36	0.23

* No weir height increase was implemented at this location.
+ Increase was negligible.

Figure 49 shows the method by which the storage volumes were calculated using the level pool estimation. Also indicated is the relative magnitude of the overestimated storage volume by not considering a level wastewater elevation. Often, in-system storage volumes are incorrectly estimated by failure to utilize the proper method. By not considering a level pool, as illustrated in Figure 49, in-system storage volumes can be overestimated by a factor of two and, depending on sewer slope conditions, by a greater amount (30).

Rationale for Regulator Modifications--

To provide for maximum inflow into the St. Paul Boulevard Interceptor (SPBI), thereby minimizing CSO discharged to the Genesee River, the trunk sewer regulators required modification. Each regulator was evaluated with respect to maximum potential discharge, resulting overflow discharge rates, and ability of the SPBI to accept increased hydraulic loads without inducing detrimental surcharge and backwater conditions in the interceptor. All outlined recommendations represented minimal modifications in adherence to the concept of Best Management Practices.

The objective of implementing regulator modifications was to increase the rates of inflow to the SPBI such that the interceptor would flow full during storm events but become surcharged. Modifications would be made, however, to accept the most inflow from those service areas with the highest pollutant loadings as determined by sampling of the CSO discharges.

It must be noted that the regulator modifications were effective in reducing CSO's because of the underutilization of conveyance capacity of various sections of the SPBI. The first assessment in determining the feasibility of minimal regulator modifications was a hydraulic analysis of the SPBI. Once this evaluation indicated that additional wastewater rates could be conveyed

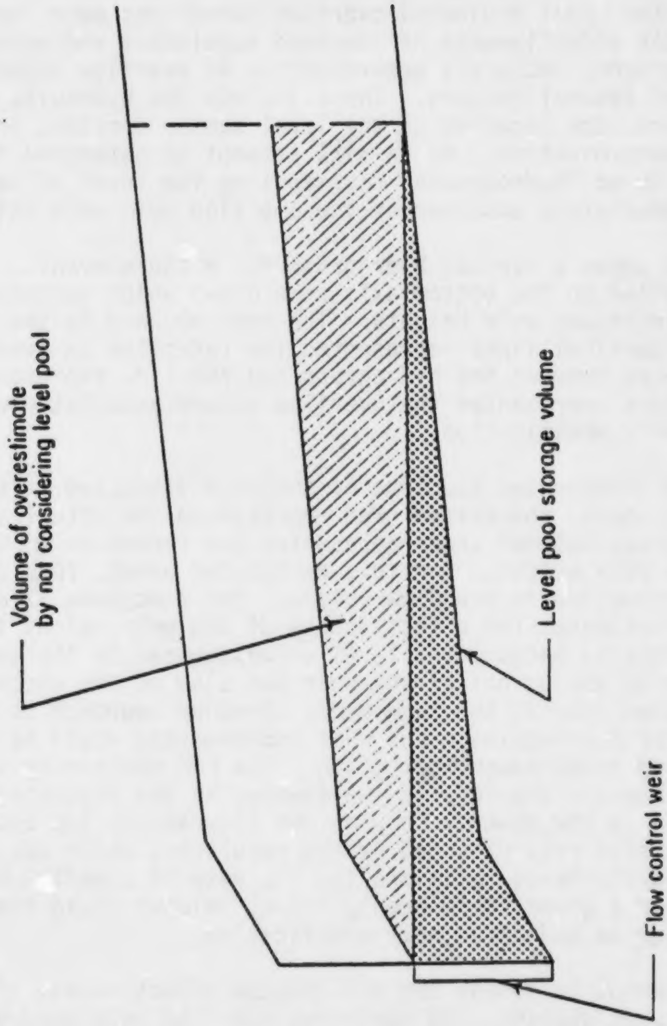

Volume of overestimate
by not considering level pool
Level pool storage volume
Flow control weir

Figure 49. In-system storage volume estimations using the level pool method.

by the interceptor, a review was made of the regulators to determine the extent of such modifications necessary to affect an increase in transfer rate.

Regulator/Weir Modification Effectiveness--

The effect on annual overflow reduction as a result of the realization of in-system storage in conjunction with improved system regulation was presented in Table 37. Condition 3 involved the application of those BMP measures such as minimal regulator and weir modifications.

Although the model projected overflow reductions were impressive, the true test of the effectiveness of improved regulators and weirs was actual measured reductions. Accurate determination of overflow reductions was difficult because of several factors. These include the hydraulic conditions in the trunk sewers, the location of the level sensor monitor, and the equations used in flow determination. An initial attempt to determine the actual reductions, monitored "hydrographs" representing the level of wastewater in the trunk sewer immediately upstream of the overflow weir were utilized.

Figure 50 shows a typical hydrograph for a storm event. As indicated, two lines parallel to the bottom axis were drawn which represented the orinal and the increased weir heights. The area bounded by the hydrograph between the two parallel lines represented the reduction in overflow volume; whereas, the area between the hydrograph and the line representing the original weir height represented the overflow volume associated with the site prior to the weir modification.

Figure 50 illustrates the type of analysis discussed in the previous paragraph. As shown, the shaded area represented the actual overflow volume; whereas, the cross-hatched area represented the reduction in CSO volume by an increase in weir height. For this particular event, this procedure indicated an 88% reduction in overflow volume. The procedure illustrated in Figure 50 overestimates the effectiveness of the weir height increase in reducing CSO. This is because the level of wastewater in the sewer is not dependent only on the height of the weir but also on the wastewater flowrate and the diversion rate at the regulator. Another approach to determining the effectiveness of the regulator and weir improvements would be to use the actual monitored trunk sewer hydrograph. The CSO monitoring system had been configured to measure the level of wastewater at the regulator and not the actual flowrate in the sewer. Knowing the flowrate in the trunk sewer and wastewater transfer rate provided by the regulator, which was approximated by an orifice head/discharge relationship, the rate of overflow could be easily determined. For a given storm event, actual volumes could then be computerd for events prior to and after weir modification.

Another method to assess the anticipated effectiveness of the weir modifications was used whereby, the monitored overflow data base was utilized. A statistical regression analysis was performed on the overflow data for each site for those events occurring prior to after implementation of the weir modifications. The general trend of the results indicated that without improvements to the SPBI, as previously identified, that the regulator and weir modifications as implemented under the BMP program reduced annual overflow to

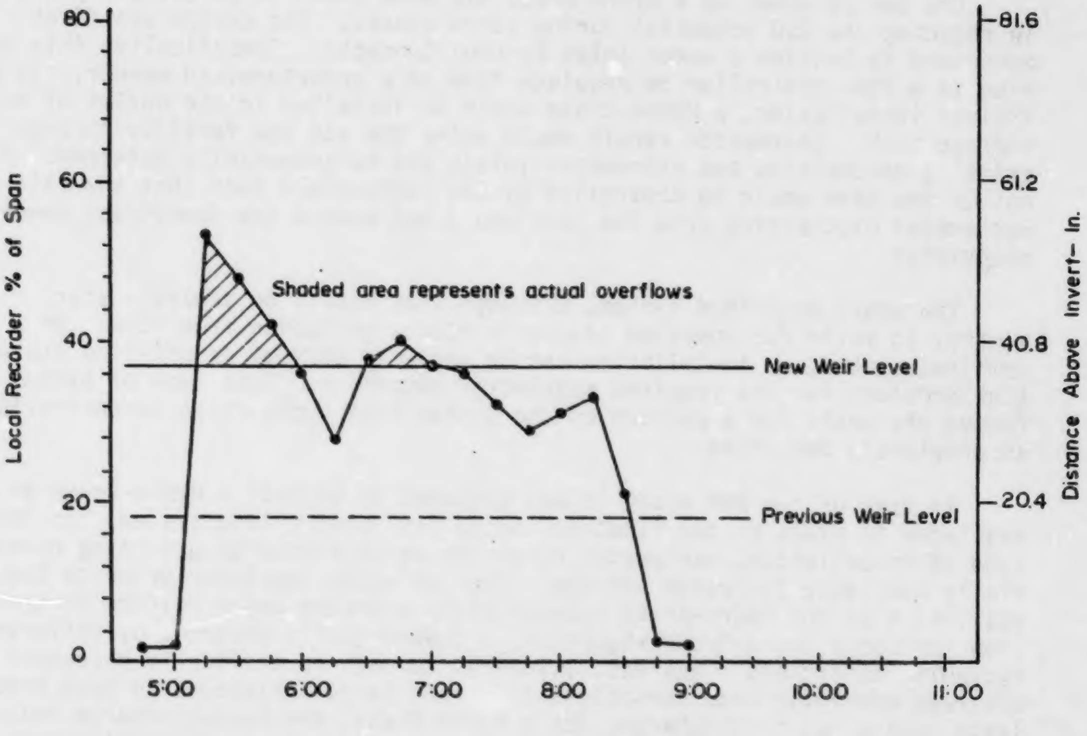


Figure 50. Example of effectiveness of increasing overflow weir heights.

the Genesee River by approximately 5-10%. This is in agreement with the model projected results which indicated about a 7% reduction.

This reduction of 5-10% was significant because the regulator and weir modifications were implemented at practically no capital expense. That is, approximately a 10% reduction was realized by simply "fine tuning" the conveyance system. The actual effectiveness that could be realized by other municipalities depends on specific system conditions. Such factors as existing interceptor and trunk sewer capacities, regulator capabilities and ease of modification, and various land use parameters will determine the actual effectiveness of such minimal improvements which fall under the general concept of BMPs.

Hydro-Brake Regulator Evaluations

Background--

The device known as a Hydro-Brake has been shown to be extremely useful in reducing the CSO potential during storm events. The device was first described in Section 6 under Inlet Control Concepts. Specifically, this device is a flow controller to regulate flow in a predetermined manner. In a typical installation, a Hydro-Brake would be installed in the outlet of a storage tank. Stormwater runoff would enter the storage facility through existing catchbasins and stormwater inlets and be temporarily detained. Flow out of the tank would be controlled by the Hydro-Brake such that the rate of wastewater discharging from the tank would not exceed the downstream sewer capacities.

The above described system, although used mostly on separate storm sewers, is valid for combined sewers to reduce pollution from CSOs. On a combined system, an installation can be designed whereby an off-line storage tank provides for the required wastewater detention. This type of system formed the basis for a portion of the Santee Area Hydro-Brake Demonstration as previously described.

As part of the BMP study it was proposed to utilize a Hydro-Brake as a regulator in place of the float-activated flow controlling system. In this type of installation, wastewater detention was achieved by utilizing potentially available in-system storage. The Lexington regulator prior to the installation of the Hydro-Brake hydraulically operated under orifice control. Flow through a Hydro-Brake regulator is theoretically governed by different hydraulic conditions. Whereas, discharge through an orifice is dependent on upstream hydraulic head conditions such that large variations in head produce large variations in discharge, for a Hydro-Brake, the head-discharge relationship is such that large changes in head cause relatively small changes in discharge. Thus, a system can be designed whereby the wastewater transfer rate can be established so that the downstream sewer is not surcharged. The backed-up wastewater would be temporarily stored in the trunk sewer.

Site Selection and Installation--

For this study an overflow site associated with a large tributary drainage basin was modified by the installation of a Hydro-Brake regulator. It was essential that the trunk sewer leading to the regulator have in-system storage. Furthermore, a site had to be chosen where the Hydro-Brake would be easily installed relative to site conditions.

After a review of the potential sites suitable for installation, the Lexington Avenue overflow location was selected. This site was suitable because of the large upstream drainage area, potential for in-system storage.

Work began in January, 1980 with the removal of the steel sleeve in the wall of the existing float-activated regulator. The orifice plate and pivot arm and chain associated with the float were removed and the opening made larger. Once the opening was sufficiently large, the Hydro-Brake was lowered into the Lexington Avenue trunk sewer through an upstream access shaft, carried downstream to the regulator, inserted into place, and quick-setting concrete placed around the unit to seal and fix the Hydro-Brake in its proper position. During the construction, as much of the work as possible was done with flow still discharging through the existing regulator. This procedure minimized the amount of wastewater that had to be bypassed to the Genesee River.

The Lexington Avenue Hydro-Brake unit is shown in Figure 51. The unit is 42-in. in diameter at the rear and tapers down to 18-in. toward the discharge end. Flow in the cunette of the Lexington Avenue sewer was routed into the Hydro-Brake by re-forming the bottom of the tunnel to fit the influent weir opening. When the trunk sewer flow exceeds the capacity of this opening, the wastewater level rises in the sewer. Flow through the Hydro-Brake is governed by the head-discharge relationship presented in Figure 52. This curve represents the design head-discharge relationship. Actual measurements were not available for accurate verification. In conjunction with the installation of the Hydro-Brake, the overflow weir at this site was raised to realize the potential in-system storage in the trunk sewer.

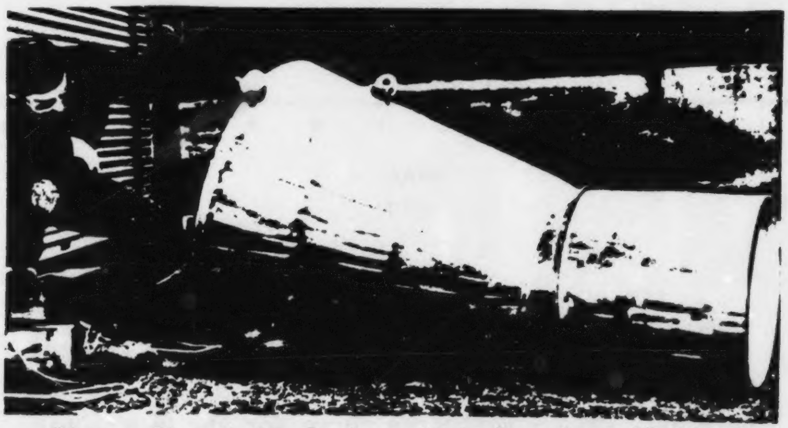


Figure 51. Hydro-Brake unit before installation.

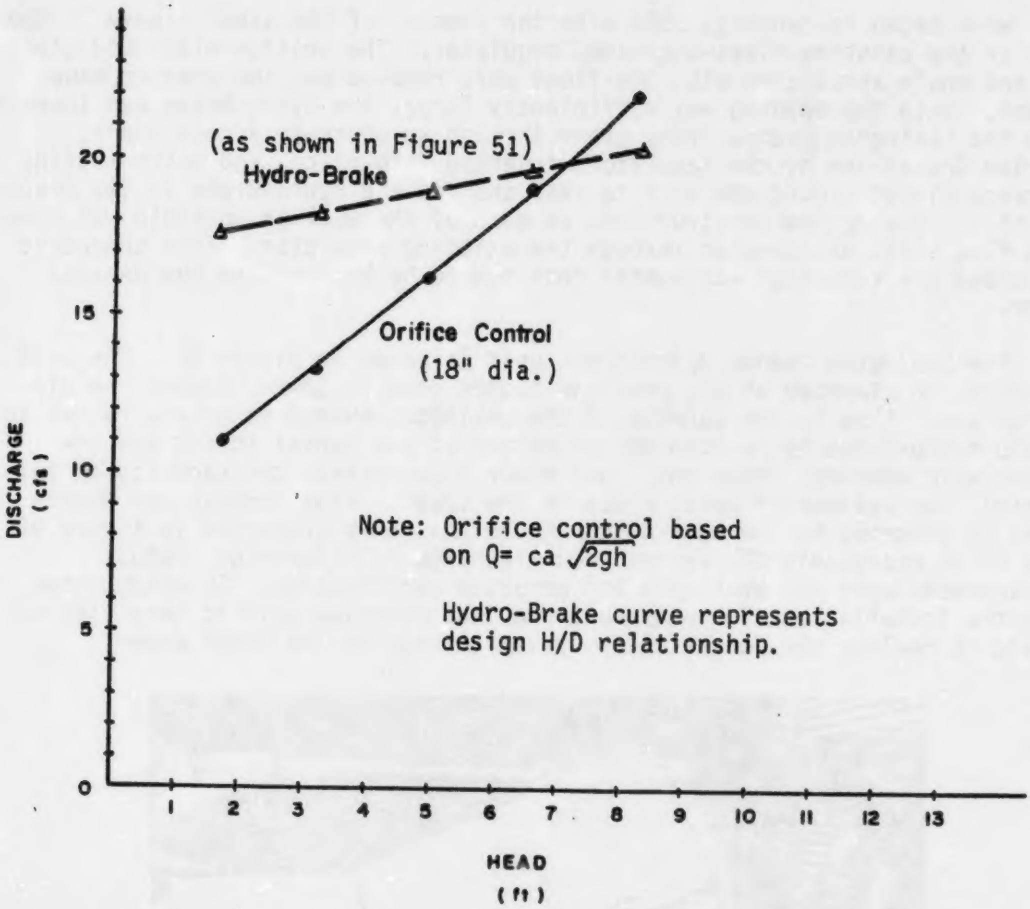


Figure 52. Head - discharge curve for the Hydro-Brake regulator at Lexington Avenue.

Operation and Maintenance Requirements--

The Hydro-Brake is constructed of stainless steel and therefore should resist corrosive sewer environments. Having no moving parts allows for operation without constant maintenance typically required of mechanical devices. The Hydro-Brake is also supposed to be a non-clogging device. In 10 months of operation this appeared to be the case. Partial clogging lasting about two weeks was observed during the fall of 1980. Before an inspection could be conducted, an intense storm occurred. During the resulting high flowrates in the trunk sewer, the clogging material was scoured out. Figure 53 shows the Hydro-Brake regulator installed at Lexington Avenue looking at the inlet to the unit.

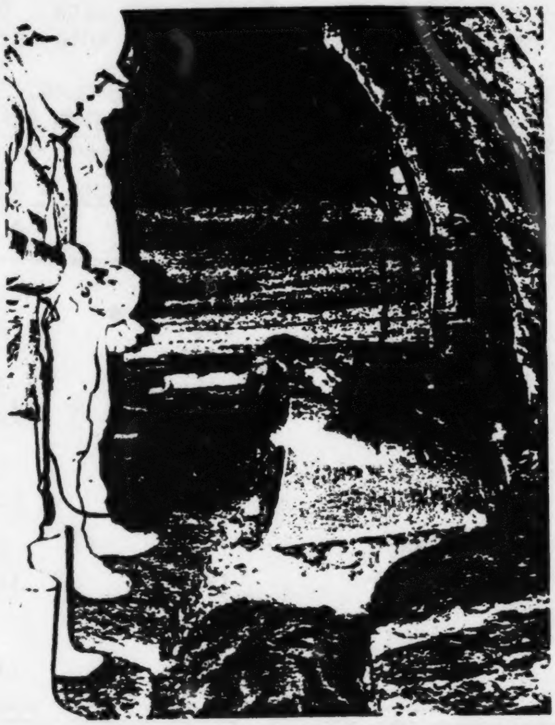


Figure 53. Photograph of installed Hydro-Brake regulator looking at the inlet.

Anticipated Overflow Reductions--

An analysis was conducted using the Simplified Stormwater Model (SSM) to estimate the effectiveness of the Hydro-Brake regulator in reducing overflow to the Genesee River. Various combinations of storage volumes and treatment rates were input into the model. Output consisted of annual CSO volumes and durations for the various input conditions. An estimate was also made of the potential in-system storage that was realized by increasing the height of the overflow weir.

Included in the model input data base were the total drainage area in acres, a gross runoff coefficient, available in-system storage, and the wastewater transfer rate from the Lexington Avenue trunk sewer to the interceptor. The area involved was 710 ac with an approximate overall runoff coefficient of 0.41. In-system storage varied from 0 to 0.50 MG and the transfer rate varied from 0.01 to 0.40 MGH. These particular ranges for storage and transfer rate were selected because they represented values ranging from present conditions to somewhat beyond those resulting from weir height increases and the installation of the Hydro-Brake regulator.

Table 43 presents the overflow volumes and durations for various combinations of in-system storage and wastewater transfer rates. Figures 54 and 55 represent graphical presentations of the same data. Overflow was based on an average annual rainfall year as determined by total precipitation.

TABLE 43. REDUCTION IN OVERFLOW VOLUME AND DURATION FOR VARIOUS STORAGE/TREATMENT COMBINATIONS AT THE LEXINGTON AVENUE REGULATOR

	Transfer Rate - MGH			
Storage - MG	0.01	0.10	0.25	0.40
0	235 (693)	178 (425)	127 (240)	95 (143)
0.1	220 (636)	164 (360)	117 (215)	88 (131)
0.2	211 (591)	154 (322)	109 (192)	82 (121)
0.3	207 (562)	146 (285)	102 (171)	76 (110)
0.5	204 (552)	132 (257)	91 (148)	67 (88)

Note: Values are average annual overflow volume in MG, whereas, values in parantheses are overflow durations in hours.

The results of the modeling indicated the following:

- Prior to the weir height increase and the installation of the Hydro-Brake regulator, the Lexington overflow regulator discharged approximately 230 MG of CSO with a duration of almost 700 hrs. Before implementation of these control measures there was essentially no in-system storage and the float-activated regulator allowed for about 0.01 MGH in terms of transfer rate.

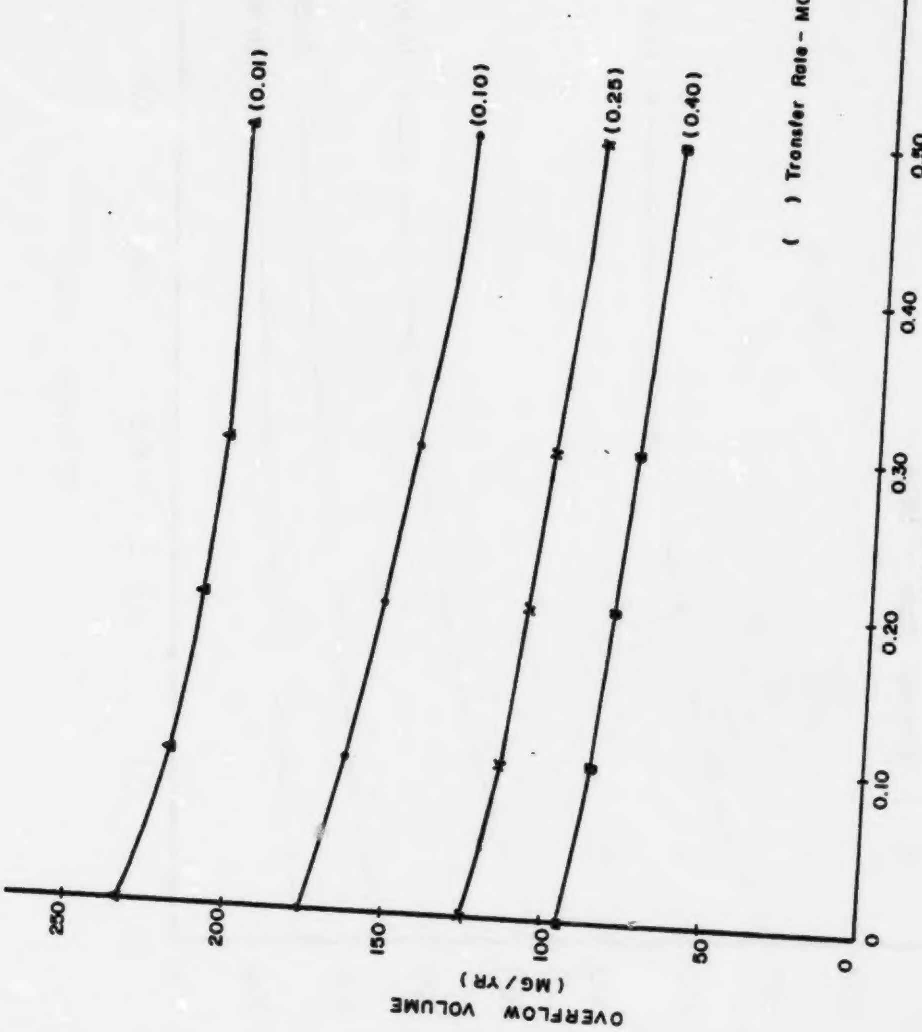


Figure 54. Overflow volume vs. storage/treatment relationship for Hydro-Brake regulator at Lexington Avenue.

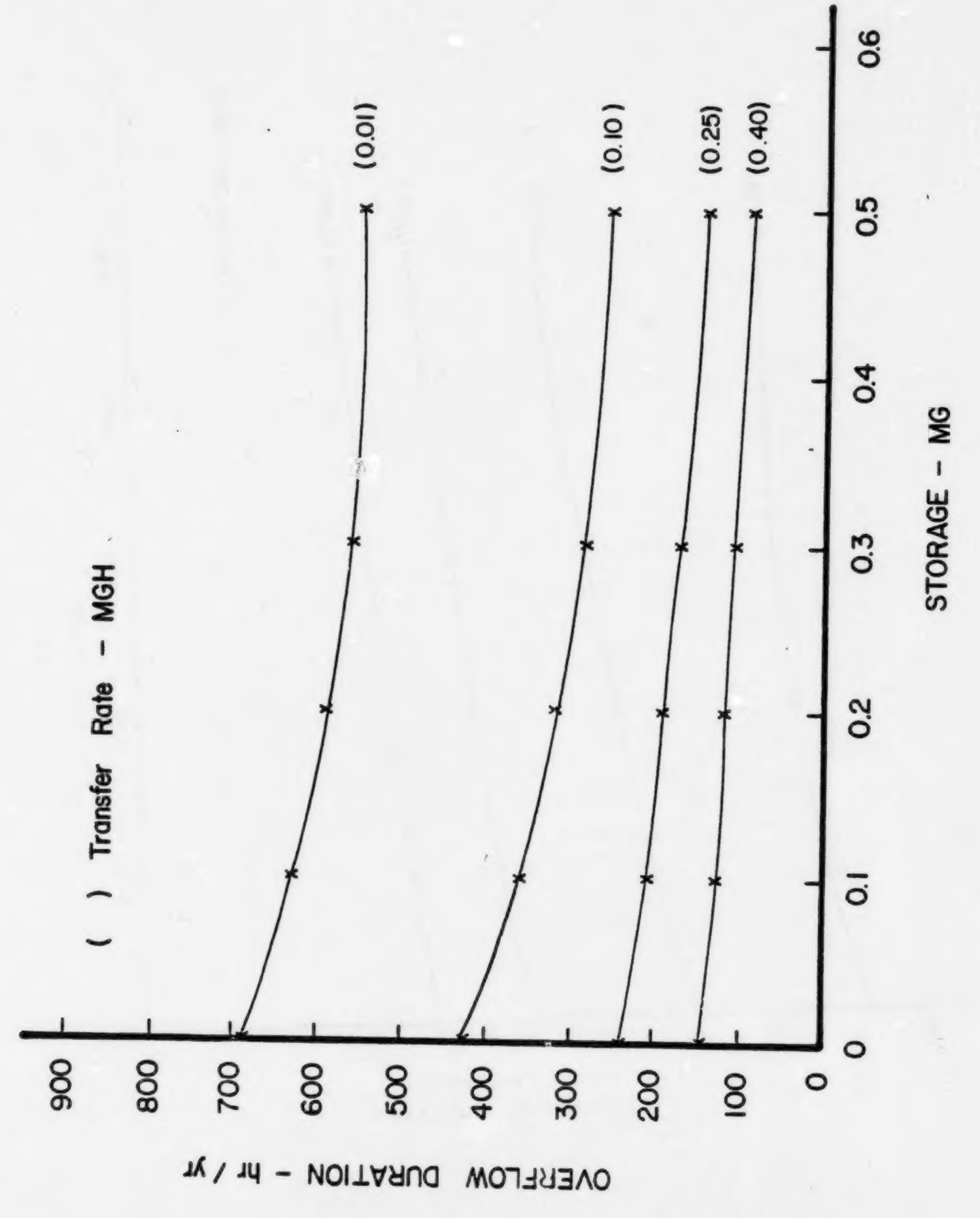


Figure 55. Overflow duration vs. storage/treatment relationship for Hydro-Brake regulator at Lexington Avenue.

- For the ranges in storage and transfer rate shown, an increase in the transfer rate yielded a much larger reduction in overflow volume and duration than an increase in storage.

- Increasing the utilization of potential in-system storage did not have a pronounced effect on reducing the average annual volume and duration of CSO.

- Significant reductions in CSO were better achieved by increasing the wastewater transfer rate than by increasing in-system storage.

- Reduction in overflow duration occured at a faster rate than reduction in overflow volume for increases in transfer rate and storage.

- Based on an estimated in-system storage volume of 0.11 MG as a result of the weir height increase and an increase in the transfer rate to about 0.25 MGH due to the installation of the Hydro-Brake regulator, the average annual overflow volume and duration were reduced by 51% and 69%, respectively.

An effort was made to field calibrate the Hydro-Brake unit and to demonstrate its effectiveness by measuring wastewater levels upstream and downstream of the regulator. A Manning ultrasonic level recorder had been previously installed at the Lexington Avenue regulator to determine the frequency and magnitude of combined sewer overflow discharges. A portable Manning ultrasonic unit was installed in the existing regulating chamber immediately downstream of the Hydro-Brake unit. Although flowrates upstream and downstream of the Hydro-Brake cannot be accurately determined with the level monitors because of turbulent flow conditions, recorded wastewater levels were directly related to discharge rate. That is, quantitative head/discharge relationship cannot be established but a qualitative assessment can be made.

Figures 56 and 57 show the relationship between the upstream depth in the Lexington Avenue tunnel and the depth downstream of the Hydro-Brake unit. These figures show that the Hydro-Brake regulator controls the discharge rate from the trunk sewer to the interceptor in the manner in which the unit was designed. That is, large increases in depth in the trunk sewer are not reflected in the wastewater transfer rate to the interceptor as would be the case if the regulator operated under orifice control. Additional head/discharge curve verification is required.

SELECTIVE TRUNK SEWER INVESTIGATIONS

General

Closely associated with improvements to the SPBI and improved system regulation were the inspections of the West and East Side Trunk Sewers. Although only a small percentage of the entire trunk sewer system for the City of Rochester consists of unfinished rock tunnels, these tunnels represent large diameter sewers. Substantial in-system storage could possibly be

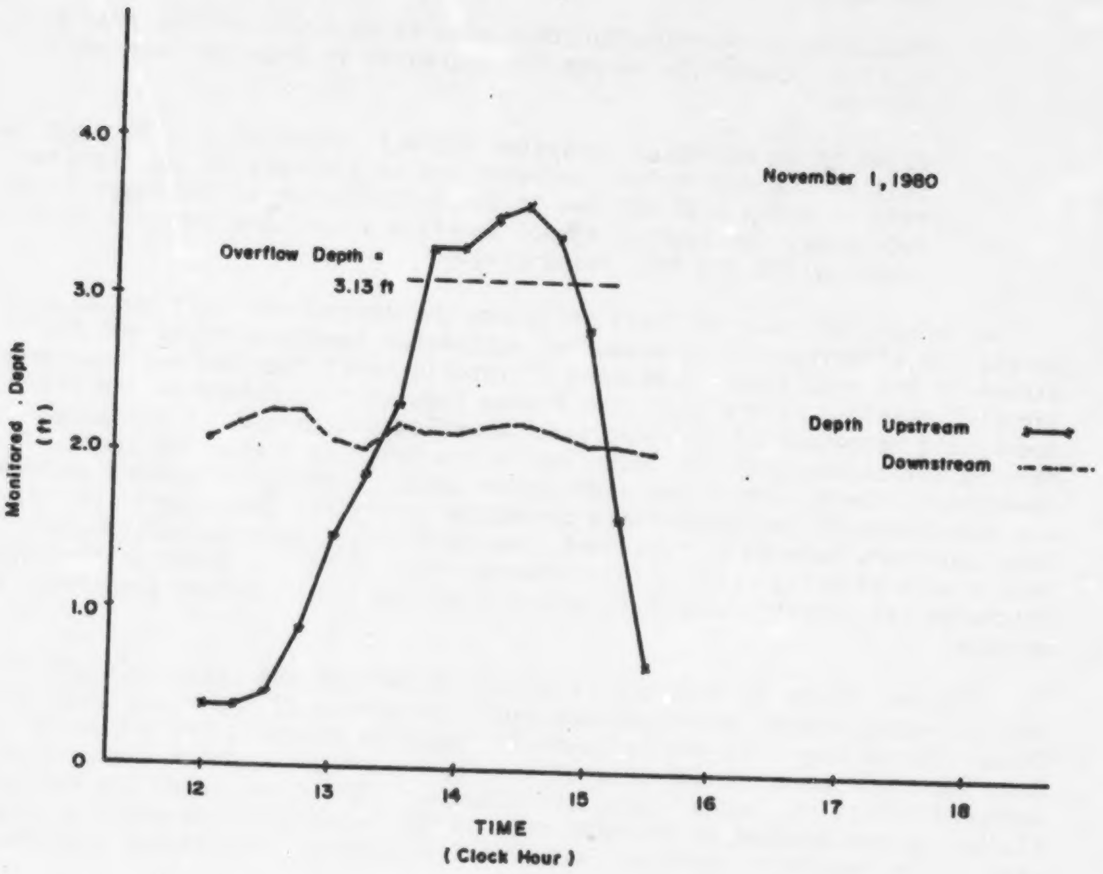


Figure 56. Relationship between upstream and downstream depths at the Lexington Avenue regulator for the 11/1/80 storm.

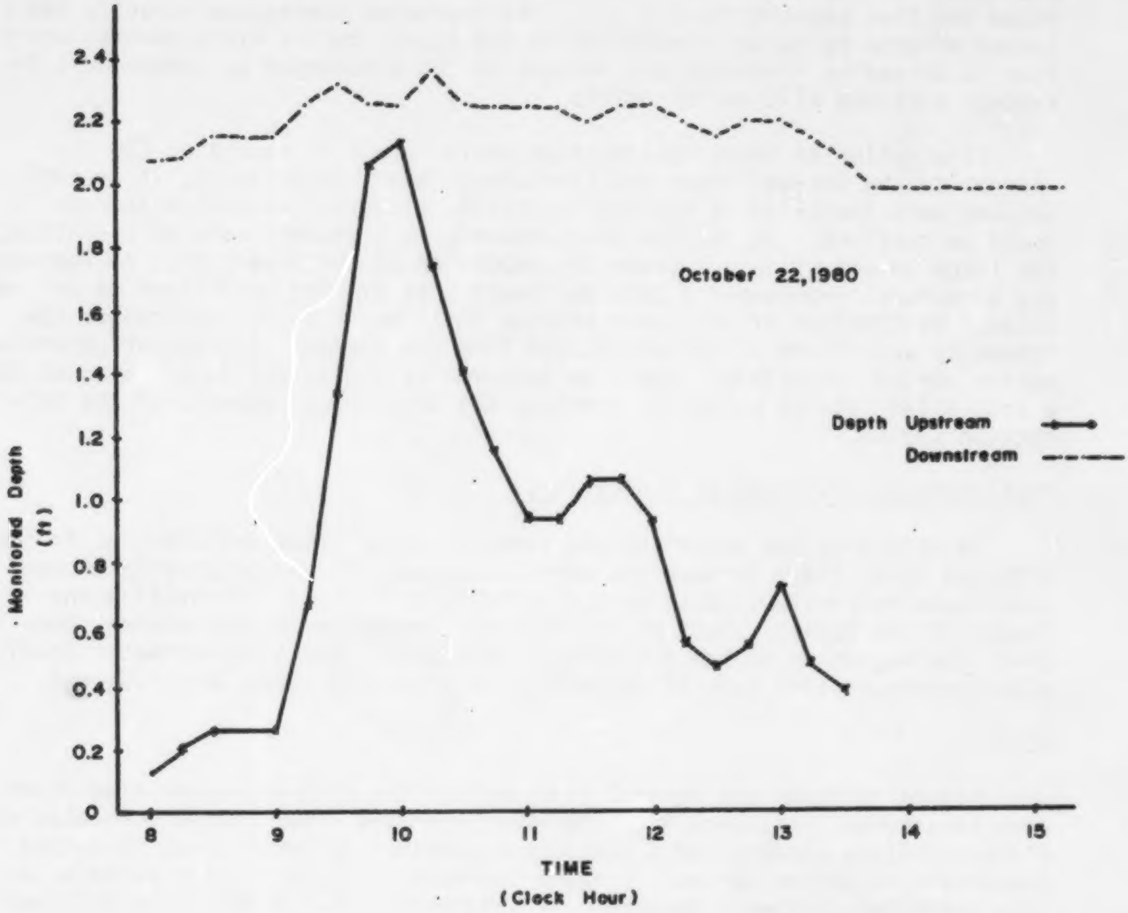


Figure 57. Relationship between upstream and downstream depths at the Lexington Avenue regulator for the 10/22/80 storm.

realized if control structures were installed at various locations within the system and if structural rehabilitation of portions of these tunnels, where necessary, was constructed.

Many sections of the WSTS and ESTS were originally constructed by blasting through rock and were left as unfinished rock conveyance tunnels. Over the years many tunnel sections have deteriorated to the point of partial roof collapse (6, 8). Material dislodged from the top of these tunnels has accumulated to the point where significant reductions in conveyance capacity have occurred. Estimated debris deposited in the ESTS along Norton Street has reduced its flow capacity by 50% (6). The decreased conveyance capacity has caused adverse backwater conditions in the tunnel during storm events, which lead to excessive frequency and volumes of CSO discharged to Irondequoit Bay through upstream ESTS relief points.

Elimination of these restrictions would result in decreased CSO discharges to the Genesee River and Irondequoit Bay. In addition, if control devices were installed at various locations, potential in-system storage could be realized. To utilize these tunnels in a storage mode of operation, the large amount of debris presently deposited at the invert must be removed and structural improvements made to insure that similar conditions do not reoccur. Utilization of in-system storage would serve to further reduce the frequency and volume of CSO discharged from the system. Stormwater generated during periods of rainfall could be temporarily stored and later released at a controlled rate so as not to overload the downstream segments of the collection system.

Field Surveys & Structural Evaluations

To determine the magnitude and severity of reported deficiencies in the WSTS and ESTS, field inspections were conducted. It was originally thought that these inspections could be accomplished by a field crew walking the length of the tunnel. Most of the WSTS was inspected in this manner, however, the magnitude of the accumulated debris and resulting backwater conditions precluded this type of inspection for the ESTS along Norton Street.

WSTS--

Figure 58 shows the general alignment of the WSTS and associated diversion structures. Entrance into the sewer was made from a manhole located at Alice and Glide Streets and a continuous physical inspection was conducted downstream to Santee Street. A sharp increase in grade at this manhole location prevented further inspection. Approximately 9800 feet of the WSTS was inspected. Approximately 3600 feet of the tunnel portion, between Santee Street and Glenwood Avenue, was not walked. Within the unwalked section of the WSTS, a lamping inspection at several manholes indicated no major structural failures or significant sediment accumulation. Although some minor deposits of rock and grit were observed at various locations throughout the walked portion of the WSTS, no significant structural defects, obstructions, or sediment accumulations were observed. Table 44 indicates the specific

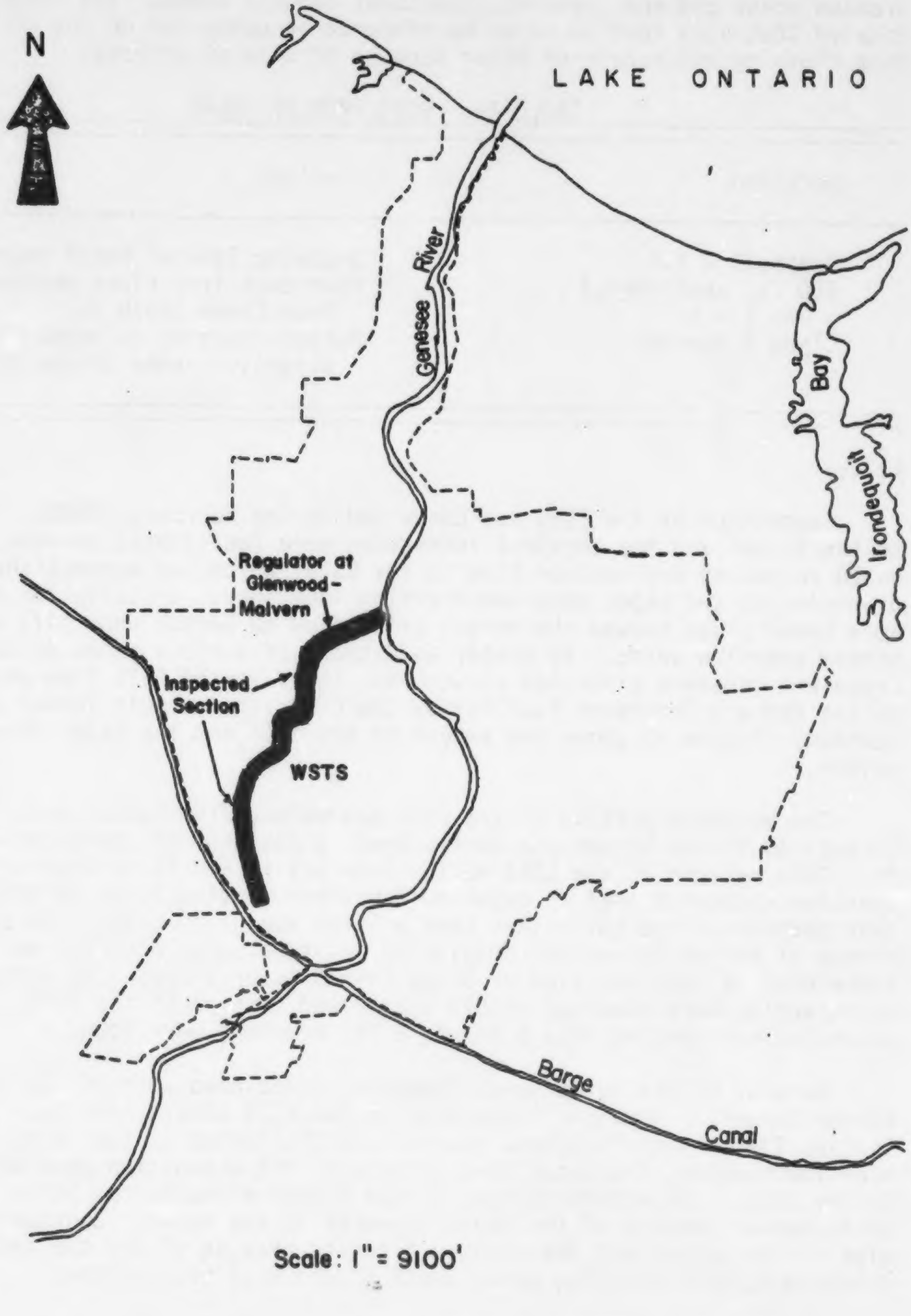

N
LAKE ONTARIO
Genesee River
Irondequoit Bay
Regulator at Glenwood-Malvern
Inspected Section
WSTS
Barge Canal
Scale: 1" = 9100'

Figure 58. Location map for West Side Trunk Sewer

problem areas and the types of structural defects noted. The inspection indicated that only routine sewer maintenance be conducted on the WSTS including cleaning and repair of minor manhole structural defects.

TABLE 44. WSTS PROBLEM AREAS

Location	Problem
Campbell & R.R.	Incoming lateral needs repair
200 ft. upstream of Jay & R.R.	Four cast iron pipes protruding from Crown 16-18 in.
Clyde & Burrows	Bottom starting to break up slightly - some bricks missing

ESTS--

Inspection of the ESTS was conducted during February, 1980. Access into the tunnel and the physical inspection were facilitated because of the reduced volume of dry-weather flow in the ESTS, which was accomplished by flow diversion at two major upstream overflow locations. Installation of inflatable sewer plugs caused the normal sewer flow to backup and spill over upstream overflow weirs. By proper adjustment of various gates at the Thomas Creek and Densmore diversion structures, the diverted ESTS flow was conveyed to the VanLare Treatment Facility by the Cross-Irondequoit Tunnel and Pump Station. Figure 59 shows the extent of the ESTS and the major diversion points.

The upstream portion of the ESTS was walked from Rocket and Edgeland Streets to Norton Street and Waring Road, a distance of approximately 5900 ft. This section of the ESTS varies from 5.5 to 6.0 ft in diameter. The inspection indicated that no major obstructions or structural defects exist in this section of the ESTS other than a large amount of grit. The ESTS upstream of Norton Street was originally constructed by open-cut and consists, therefore, of concrete pipe or brick formed sewer shapes. No structural deficiencies were observed within the walked portion of the ESTS. Grit accumulations ranging from 6 in. to 2 ft, however, were found.

Because of the large sewer diameters associated with the ESTS along Norton Street, a physical inspection of the ESTS should have been conducted. Initial field investigations, however, which started at the Jewel Street overflow chamber, indicated that structural and deposition problems do exist in the ESTS. The entire stretch of the tunnel along Norton Street could not be inspected because of the depth of water in the sewer. Considering the size of the tunnel and the reduced flowrate because of the two upstream diversions, the resulting water depth should have been minimal.

From the Waring Road chamber at Jewel Street the ESTS is an unfinished (unlined) rock cut tunnel constructed about 87 years ago. At several locations along the ESTS tunnel a portion of the crown has fallen leaving a large

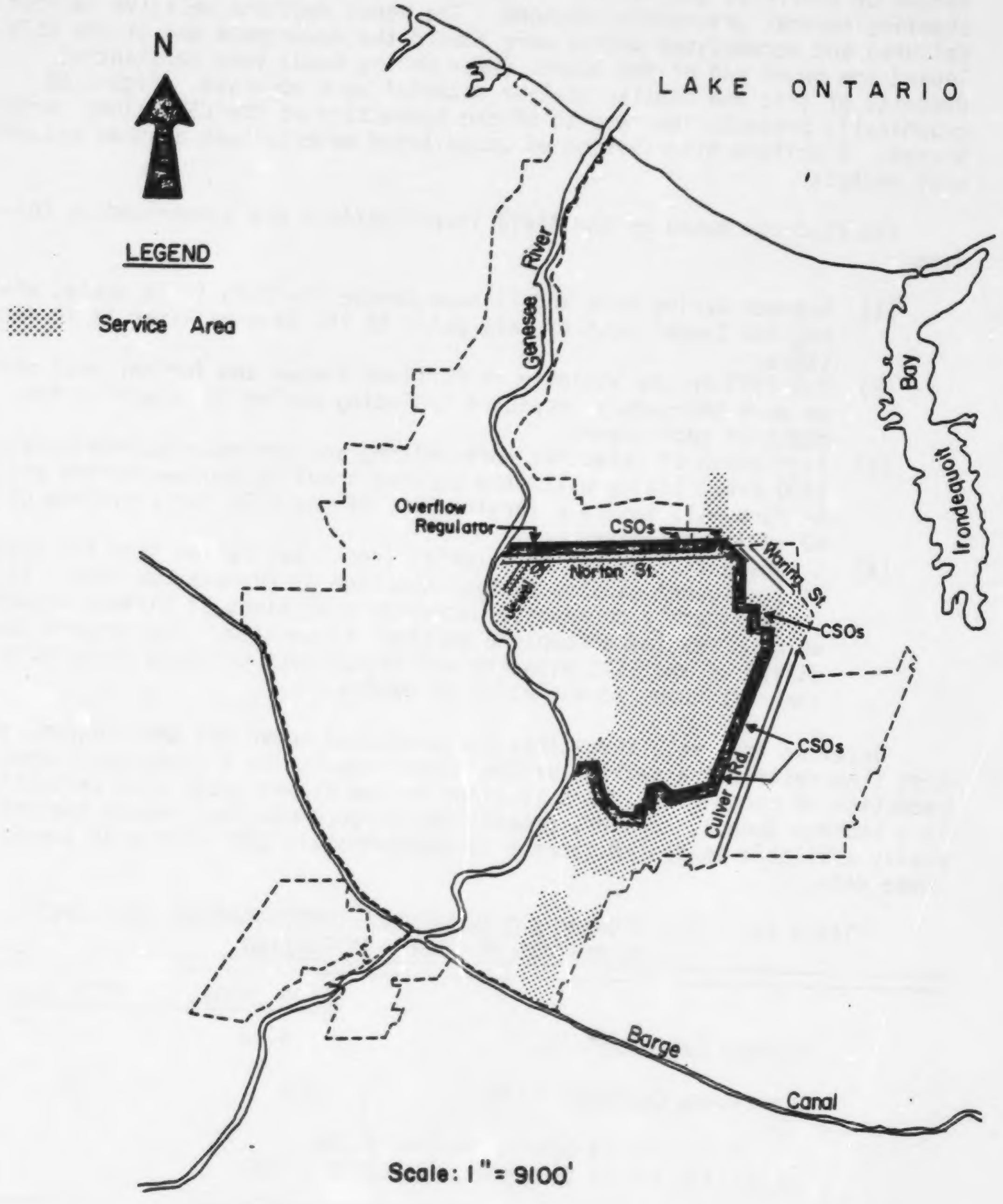

N
LAKE ONTARIO
LEGEND
Service Area
Genesee River
Irondequoit Bay
Overflow Regulator
CSOs
Norton St.
Waring St.
CSOs
CSOs
Culver Rd.
Barge Canal
Scale: 1" = 9100'

Figure 59. Location map for East Side Trunk Sewer

amount of debris at the invert. This debris is causing the flow to pond creating several impassable sections. The worst sections relative to roof failures and accumulated debris were toward the downstream end of the ESTS. Toward the upper end of the tunnel (near Waring Road) very substantial deposits of grit and similar smaller material were observed. Figure 60 graphically presents the results of the inspection of the ESTS along Norton Street. A uniform distribution of accumulated material was assumed between each manhole.

The findings based on the field investigations are summarized as follows:

(1) Between Waring Road and Clinton Avenue the ESTS is in shale, whereas, the tunnel west of this point to the Genesee River is in sandstone.
(2) The ESTS in the vicinity of Portland Avenue and further east should be more thoroughly inspected including coring to determine the depth of rock cover.
(3) Initiation of selective rock-bolting and concrete (approximately 1700 feet) lining will stop current spauling and weathering which in turn will insure a service life of the ESTS for a minimum of 50 additional years.
(4) A substantial amount of material (rock) has fallen from the crown of the tunnel causing partial blockage of dry-weather flow. This leads directly to adverse backwater conditions at various locations which prevented a complete physical inspection. The present condition of the ESTS with the unfinished rock surfaces contribute to the continuing accumulation of debris.

Based on the field investigations conducted under the BMP program, the most flow-restrictive section of the tunnel results in a conveyance capacity reduction of about 65%. The ESTS along Norton Street could also be utilized as a storage tunnel, however, present debris accumulations reduce the potentially available in-system storage by approximately 19%. Table 45 summarizes these data.

TABLE 45. ESTS STORAGE AND CONVEYANCE CONSIDERATIONS DEVELOPED AS RESULTS OF TUNNEL INSPECTION

	CLEAN TUNNEL	WITH DEBRIS
Storage Capacity - MG	4.63	3.77
Conveyance Capacity - CFS	113	39

Reduction of Storage Volume = 19%
Reduction in Conveyance Capacity = 65%

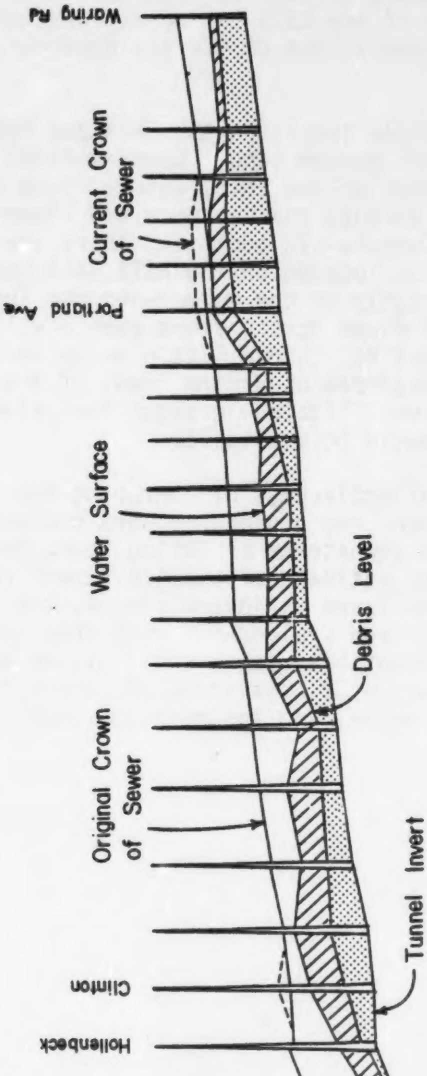

Waring Rd
Current Crown of Sewer
Portland Ave
Water Surface
Debris Level
Original Crown of Sewer
Tunnel Invert
Clinton
Hollenbeck
Jewel St

Figure 60. East Side Trunk Sewer inspection results.

Impact of Tunnel Rehabilitation on CSO Reduction

Present disposition within the tunnel portion of the ESTS contributes to more frequent CSO discharges than would occur if the tunnel were clean. The debris accumulations result in adverse backwater conditions severaly limiting the conveyance capacity of the ESTS during wet-weather events. The extent of the impact of these accumulations on CSO drainage was, therefore, investigated.

Several considerations involving the ESTS and future flow diversions to the Culver-Goodman Tunnel system (under construction) merit discussion. At the three diversion points on the ESTS, future flows during wet-weather events can be diverted through the Densmore and Thomas Creek structures into the Cross-Irondequoit Tunnel. In addition, there are many overflow relief structures proposed to be located on the ESTS, all upstream of Waring Road, that will discharge directly to the Culver-Goodman Tunnel. Because of these diversions, future ESTS flows during storm events will be lowered relative to those presently occurring. Since little or no additional wastewater flow will be in the ESTS downstream of Waring Road, if the tunnel portion of the ESTS were cleaned and rehabilitated, greater in-system volumes and increased conveyance capacities would be available.

To determine the effectiveness of improving the ESTS, evaluations using simplified stormwater modeling procedures were conducted. Based on the tributary area to the ESTS downstream of Waring Road, modeling was conducted using the total area, an estimate of average runoff coefficient, an assumed in-system storage volume based on invert slopes, the objective to minimize surcharging of the ESTS, and a bleed-off rate from the Jewel Street regulator to the SPBI which is compatible with overall system operation. Table 46 presents the model input data, consisting of combinations of storage volumes and regulator transfer rates, and the results relative to overflow frequency and volume.

TABLE 46. REDUCTION IN OVERFLOW VOLUME AND DURATION FOR VARIOUS STORAGE/TREATMENT COMBINATIONS AT THE EAST SIDE TRUNK SEWER OVERFLOW REGULATOR

Storage - MG	Transfer Rate - MGH 0.17	 0.58	 1.17	 1.75	 2.33
0	172.80 (425)	87.45 (119)	44.70 (47)	26.45 (23)	16.50 (13)
2.15	68.60 (109)	28.90 (30)	10.30 (11)	4.40 (5)	2.25 (2)
4.30	31.60 (49)	7.25 (13)	0.70 (1)	0.10 (1)	0 (0)
6.45	12.80 (27)	0 (0)	0 (0)	0 (0)	0 (0)

Note: Values are average annual overflow volume in MG, whereas, values in parentheses are overflow durations in HRS. Drainage area is 840 ac with a runoff coefficient of 0.39.

The present transfer rate was estimated to be between 0.58 and 0.17 MGH. The exact rate could be easily determined because of the deposition problems associated with the ESTS along Norton Street. Presently, there is no available in-system storage. Figure 61 shows that for no in-system storage and a transfer rate between 0.17 and 0.58, overflow volume would be about 135 MG/yr. With utilization of potential in-system storage, estimated at 2 MG and with a reduced transfer rate to 0.17 MGH to be consistent with overall system operation as discussed previously, average annual overflow would be reduced to approximately 75 MG/yr.

To fully utilize the ESTS for storage and controlled release, major rehabilitation of the unfinished rock tunnel must be accomplished. USEPA Step 1 Construction Grants assistance was applied for and received for the necessary rehabilitation work (29).

STRUCTURAL IMPROVEMENTS TO MAXIMIZE USE OF EXISTING SYSTEM

Control Structures

To utilize the large volume of potentially available in-system storage within the existing and proposed sewer system network, installation of control structures at various locations will be necessary. The storage realized by the installation of such control devices would then allow for flow attenuation, thereby decreasing the peak in-system flowrates generated during

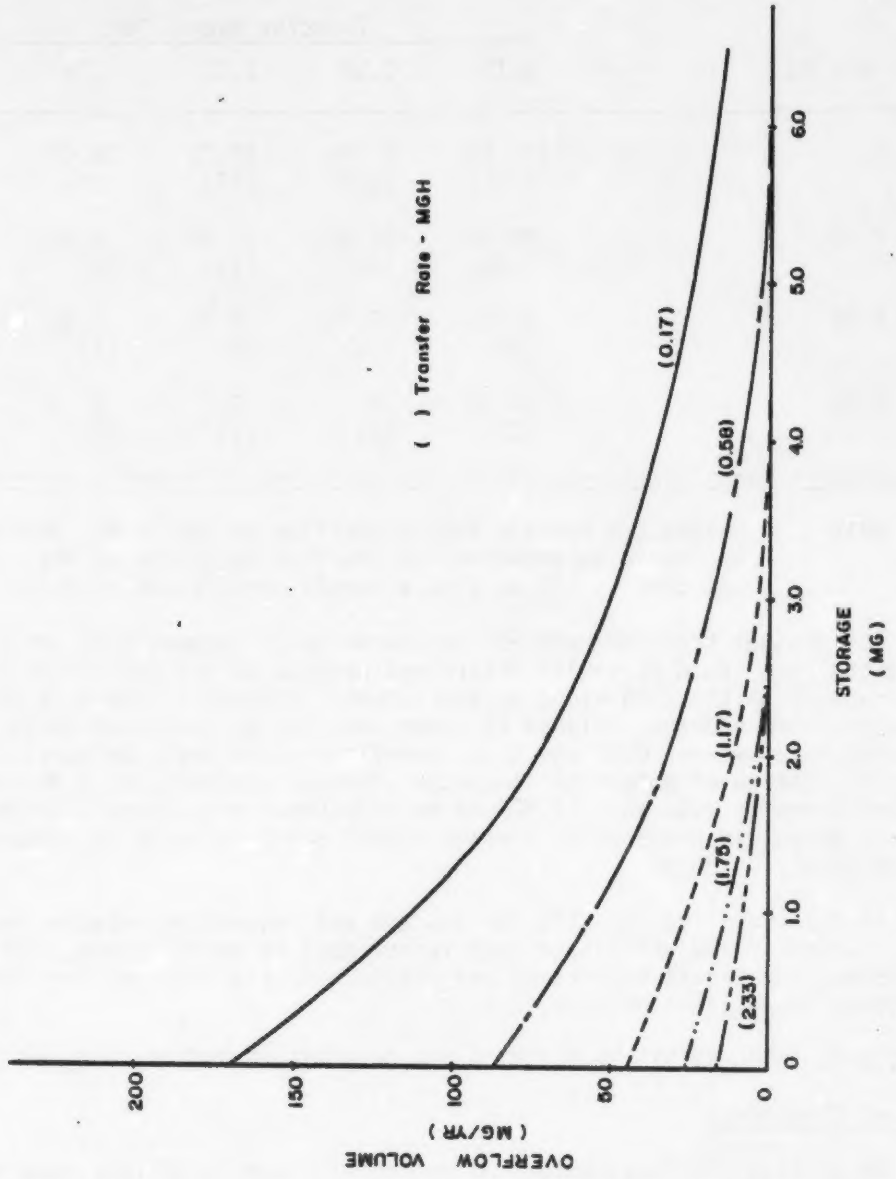


Figure 61. Overflow volume vs. storage/treatment relationships for the East Side Trunk Sewer based on simplified modeling.

storm events. The net overall result would be a decrease in the volume and frequency of CSO presently discharged from the existing sewer system.

The concept of utilizing potential in-system storage was first conceived and implemented under the regulator/weir modification effort as part of the minimal BMP system improvements. Realized in-system storage as a result of the previously identified weir height increases, although responsible for a noticeable decrease in the volume and frequency of CSO's, could be increased. More effective, positive control at selected locations within the existing sewer system was required to utilize more of the potentially available in-system storage.

Based on a review of the overall sewer network with respect to sewer sizes, shapes, lengths, and slopes, certain sewer systems within the overall sewer conveyance network for the City of Rochester were suitable for installation of control structures.

Figure 62 shows the locations of the required control structures within the existing surface conveyance system. As previously discussed, the Lexington Avenue Tunnel terminates at an overflow regulating chamber located at Lake Avenue. It is at this structure that the Hydro-Brake regulator was installed. Better control at this point would allow for greater utilization of in-system storage as shown previously in Figure 55.

The West Side Trunk Sewer (WSTS) offers significant in-system storage potential if an effective control device were installed at the overflow regulator located at Glenwood Avenue and Malvern Street. With respect to the East Side Trunk Sewer (ESTS), the concept of in-system storage utilization has been previously discussed in this Section. Figure 61 showed the effect of storage on overflow volume.

Two locations, which were not previously discussed, are Front Street and the Genesee Valley Interceptor Southwest (GVISW). Within recent years the Front Street sewer along with the immediate upstream sewer, the Main Street tunnel, were rehabilitated. They are no longer unfinished rock tunnels, but rather are lined with concrete throughout their circumference. Potential in-system storage exists in these large diameter tunnels if proper control were implemented at the Front Street overflow location.

The GVISW, which became operational in 1979, is a large diameter tunnel serving the southwest portion of the City of Rochester. It was designed and built as a storage tunnel. A potential storage volume of nearly 12 MG is available if proper control at the downstream end were implemented. A control structure exists consisting of a sluice gate designed to control the rate of outflow from the GVISW. Currently, however, there is no electric operator at this site and, therefore, if control were desired, manual opening and closing of the sluice gate would be necessary.

Based on review of the specific sewer systems upstream of each regulator, installation of control structures at the locations identified in Figure 62 would result in a minimum in-system storage volume increase of 15.15 MG. The specific volumes by location are presented in Table 47.

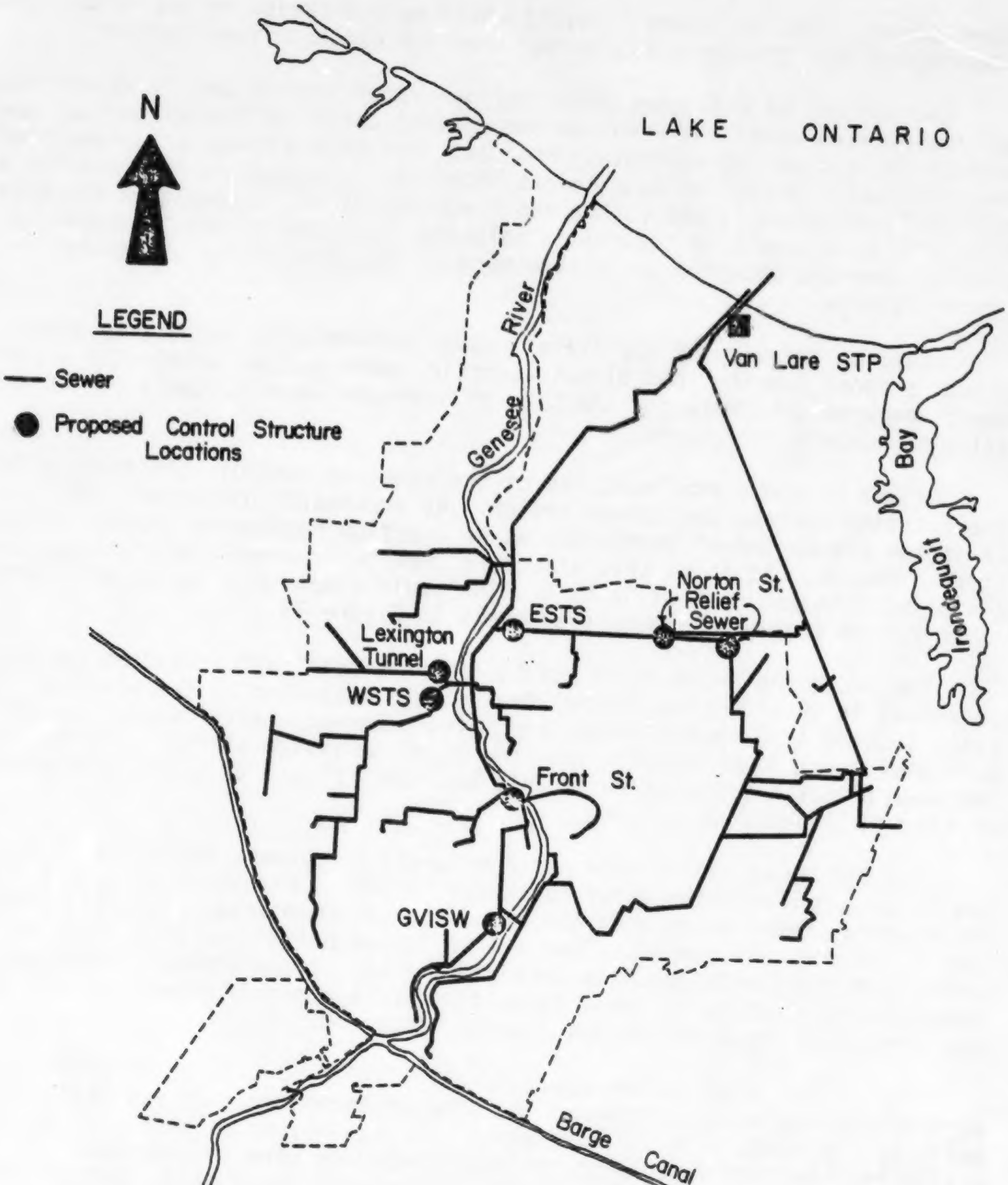

N
LAKE ONTARIO
LEGEND
— Sewer
● Proposed Control Structure Locations
Genesee River
Van Lare STP
Irondequoit Bay
Norton St. Relief Sewer
ESTS
Lexington Tunnel
WSTS
Front St.
GVISW
Barge Canal

Figure 62. Proposed control structure locations.

TABLE 47. REALIZED IN-SYSTEM STORAGE VOLUMES BY THE INSTALLATION OF CONTROL STRUCTURES

Location	In-System Storage Volume - MG
Lexington	0.11
WSTS	0.43
ESTS	2.15
Front	0.16
GVISW	12.30

These control structures represent major capital improvements to the sewer system. They are not termed minimal-structural but rather structurally intensive. Although beyond the scope of a BMP oriented program, the methodology of approach to the problem and the solution - maximizing the use of the existing system - fall under the general concept of BMP's. To implement such a program, USEPA Construction Grants funds should be applied for early in an overall BMP study effort. Funding should include monies for Step I, II, and III program phases.

Specifically, the types of control structures necessary at the identified locations within the sewer system would be electrically operated devices such as sluice gates and movable weirs. The sluice gates would provide the control necessary to throttle the rate of wastewater discharge from the storage tunnel to the downstream sewer element. The rate would be such as to prevent the possibility of overloading or surcharging a downstream section which in turn would likely result in a CSO discharge.

Standby power generation would be provided at each site so as to insure positive operation during periods of electrical power outages, which are possible especially during severe storm events occurring in the summer thunderstorm period. In any event, a fail-safe mode of operation would be installed to insure the structural integrity of the sewer system as well as to protect lives and property. That is, should all power be temporarily lost at a particular site, the control device would automatically be placed in such a position as to provide the required hydraulic relief to avoid excessive surcharging and possible structural damage.

To evaluate the effectiveness of the implementation of control devices at the identified locations, simplified modeling using the SSM was conducted. The effect on overflow volume of in-system storage was previously shown for Lexington and the ESTS. To summarize the overall effectiveness, Table 48 presents the average percent reduction in overflow volume by site for an average rainfall year.

TABLE 48. CONTROL STRUCTURE EFFECTIVENESS IN CSO REDUCTION

Location	Average Annual CSO Volume Reduction - %
Lexington	51
WSTS	14
ESTS	47
Front	44
GVISW	86

Note: These values represent minimum percent effectiveness for the installation of a control device at the downstream end of the specific tunnel involved and operated such that no surcharge occurs at this control location.

Control System

The basic objectives of this phase of the BMP program were as follows:

- To develop a plan to incorporate the overflow monitoring system into a system status component of an overall control system for CSO control and management.
- To determine the need, type, and number of rain gauges to be located throughout the City of Rochester necessary to provide the needed input data for control and management of CSO's.
- To evaluate the need, type, and location of level sensing devices to be installed at critical points in the existing sewer system to actually install the devices.
- To develop the operating logic necessary to implement and operate a real-time control system for the conveyance system existing whose present overflow points discharge to the Genesee River. Also included was the development of the basic hydraulic model to generate the required system parameters and projections to operate the overall control system.

An important aspect of incorporating the overflow monitoring system into a functional control system was the upgrading the monitoring and telemetry systems so as to function properly and reliably. As discussed in Section 5, the monitoring system, previously consisting of Badger Meter ultrasonic level and velocity systems, was replaced with Manning Corporation ultrasonic level recorders.

In addition to the replacement of the primary measuring devices, the overall telemetry system was also upgraded. Early into the BMP program centralized control was deemed preferable over local control. To provide for centralized control, telemetry had to be provided on all the major overflow monitoring sites with the data being sent to a computer centrally located. The computer, which was upgraded and modified under the BMP program, is located at the VanLare Treatment Plant. All control and status functions would be performed by one computer from this location.

The computer involved is a model PDP-8E manufactured by the Digital Equipment Corporation. Upgrading of the computer consisted of providing additional core capacity, adding a disk drive, and incorporating an additional computer display terminal. All of these modifications significantly improved the utility of the computer system to collect, store, and interpret the telemetered system status data. Figure 63 is a photograph of the computer facility dedicated to the BMP program.

After using the local rain gauge data for rainfall-runoff-overflow correlations, it was apparent that the gauges, which were provided telemetry capability under the BMP program, were not satisfactorily suited for maintaining and operating a real-time control system.

The rain gauges only recorded in 0.1 in. increments. This increment is not sufficient for the type of model input needed. The modifications that were implemented, such as regulator and weir alterations, are effective when considering small storm events. Furthermore, because of the characteristics of the sewer system, the implemented system improvements are effective over relatively small changes in rainfall amounts. Hence, the need for more accurate rain gauges. The present location of the eight local rain gauges is adequate, more sensitive recording gauges are necessary.

To adequately assess the wastewater flows throughout the sewer system network, in order to optimize its use in minimizing CSO's; it is imperative that an accurate and reliable system status monitoring system be implemented. Under the BMP program, eight level monitoring recorders were installed along the SPBI and several of the major trunk sewers. Each location was equipped with telemetry instrumentation which allowed for direct data transmission to the central receiving computer at the VanLare Treatment Plant. These monitoring locations are identified in Table 49 and are shown in Figure 64.

Figure 63. Computer facility associated with the BMP program.

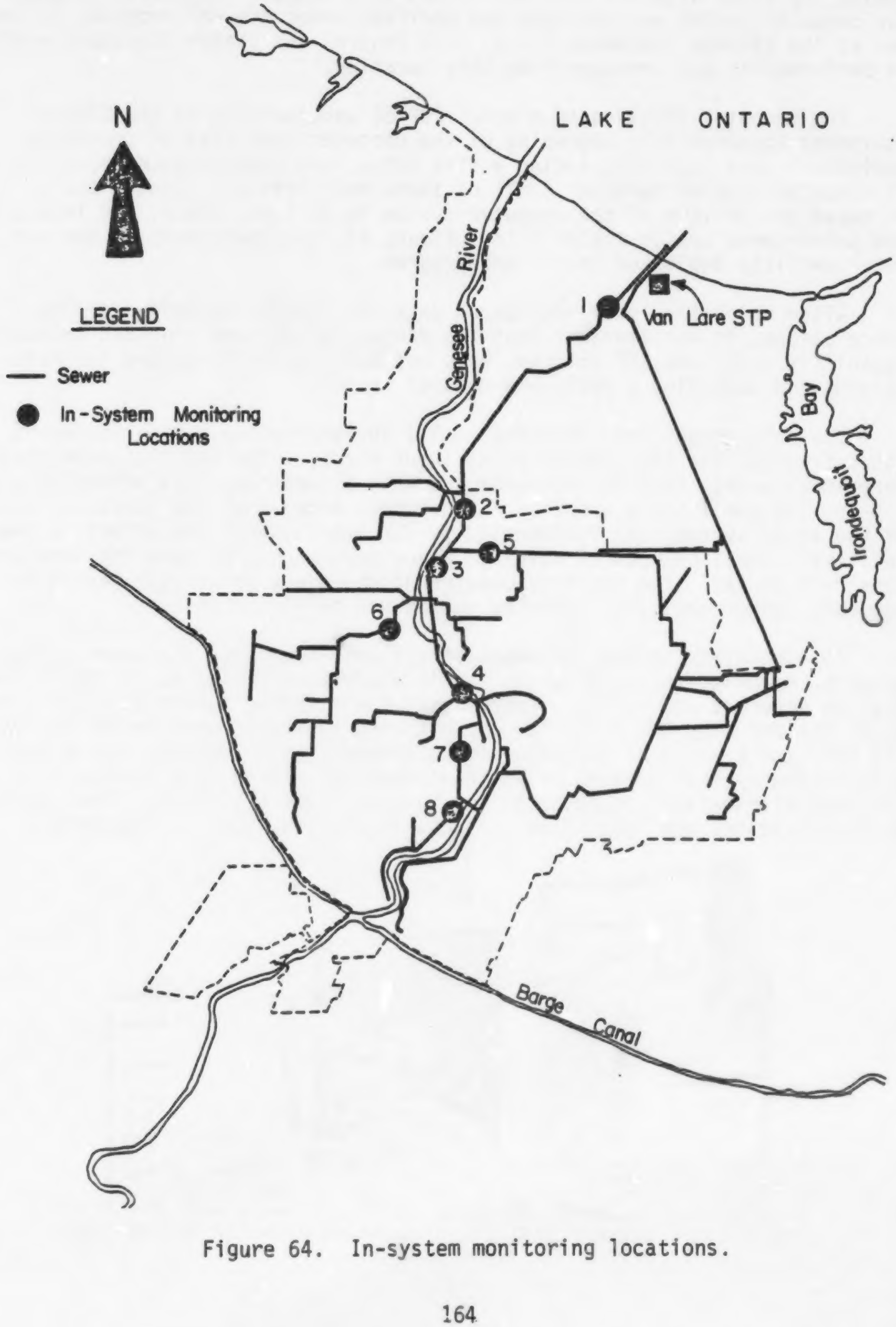

LAKE ONTARIO
N
LEGEND
Sewer
In-System Monitoring Locations
Genesee River
Van Lare STP
Irondequoit Bay
Barge Canal
1
2
3
4
5
6
7
8

Figure 64. In-system monitoring locations.

TABLE 49. IN-SYSTEM MONITORING LOCATIONS

Location	Site
Pinegrove	1
Zoo	2
Beach	3
Mill St.	4
Hollenbeck	5
Revine	6
Clarissa	7
GVISW	8

Note: Level sensor and telemetry system at the GVISW location were installed during construction of the GVISW. Only a tie-in to central computer was necessary under BMP program. All installed instrumental was ultrasonic level recorders manufactured by the Manning Corporation.

The selection of these particular sites was made on the basis of site accessibility for ease of installation and periodic maintenance, whether the trunk sewers on which they are located have significant potential in-system storage volumes, and at those locations along the SPBI, whether they would provide the needed hydraulic information for system status and control.

Although periodic problems occurred throughout the in-system monitoring program, especially with the telemetry instrumentation, the overall data collection and recording system generally worked satisfactorily and provided the necessary data. In addition to establishing the necessary status system for eventual control management, the in-system monitoring program also provided several other benefits. They were as follows:

- Providing an indication of the actual utilization of sewer conveyance and storage capacities which was essential in maximizing the use of the existing system.

- Proving that such a monitoring system utilizing ultrasonic level recorders and telemetry could provide the needed information for formulating a real-time control system.

To illustrate the usefulness of the monitored in-system data to indicate that the existing sewer system had additional capacity, Table 51 presents the maximum depth of flow at the monitored sites for selected storm events.

TABLE 50. CONTROL SYSTEM MONITORED DATA

		Rainfall				
Date	Site No.	Vol. in.	Max Hr. Intensity in/hr	Max Depth ft.	Flow MGD	Depth % of Total Pipe ø
5/13	1	0.32	0.14	2.81	98	51
5/18	1	0.40	0.12	3.19	121	58
5/31	1	0.47	0.25	2.86	101	52
6/01	1	0.40	0.32	2.86	101	52
5/13	2	0.32	0.14	3.14	96	52
5/18	2	0.40	0.12	3.35	106	56
5/31	2	0.47	0.25	3.08	93	51
6/01	2	0.40	0.32	3.14	96	52
5/31	3	0.32	0.14	3.14	54	57
5/18	3	0.40	0.12	3.14	54	57
5/31	3	0.47	0.25	3.14	54	57
6/1	3	0.40	0.32	2.89	47	51
5/13	4	0.32	0.14	5.63	108	90
5/18	4	0.40	0.12	6.00	112	96
5/31	4	0.47	0.25	6.25	103	100
6/01	4	0.40	0.32	6.25	103	100
5/13	5	0.32	0.14	3.10	15	35
5/18	5	0.40	0.12	3.46	19	40
5/31	5	0.47	0.25	3.46	19	40
6/01	5	0.40	0.32	3.81	22	44

(continued)

TABLE 50 (continued)

Date	Site No.	Rainfall Vol. in.	Rainfall Max Hr. Intensity in/hr	Max Depth ft.	Flow MGD	Depth % of Total Pipe ø
5/13	6	0.32	0.14	1.54	43	28
5/18	6	0.40	0.12	1.98	72	36
5/31	6	0.47	0.25	-	-	-
6/01	6	0.40	0.32	-	-	-
5/13	7	0.32	0.14	0.68	11	10
5/18	7	0.40	0.12	0.75	14	11
5/31	7	0.47	0.25	2.19	86	31
6/01	7	0.40	0.32	3.21	281	46

As indicated in Table 50, in most instances, the full capacity of the conveyance systems was not utilized. Review of the monitored in-system data showed that for a wide range in storm events, flowrates were generally governed by rainfall intensities and not necessarily by volume. Data reduction also showed that the regulators on the trunk sewers controlled the wastewater flow into the SPBI. That is, the SPBI under varying rainfall conditions, including very intense storms, did not flow full. Excess capacity existed throughout the interceptor. This fact formed the basis for adjusting the regulators and weirs to maximize use of the existing system.

The overall required control system must be integrated with the control system for the existing tunnel system and VanLare Treatment Plant. System status data collection must be made and sent to the central receiving computer at the VanLare STP. In this manner all system parameters such as rainfall intensities and volumes, sewer system flowrates and wastewater levels, rates and volumes of CSO's, tunnel utilization, and treatment plant operating conditions are reviewed and analyzed before control strategies are made and management control options implemented. Only then will the use of the overall conveyance and treatment systems be optimized and CSO's minimized.

All of the collection system improvements identified in this section, when implemented, will result in an integrated system approach to CSO pollution abatement. Each improvement - removal of several flow-restrictive sections of the interceptor, modified trunk sewer regulators, selective trunk sewer rehabilitation, and installation of control structures - reduces, to some degree, the volume and frequency of CSO presently discharged to the Genesee River. Because all improvements are compatible and complementary, implementation of all of the identified measures result in a substantial reduction in average annual CSO discharged to the Genesee River. Annual average overflow reduction of approximately 41% would result if all improvements were implemented.

The structural improvements to the collection system outlined in this section would be effective in reducing annual CSO because of the large number of small, low-intensity storm events which occur in any given year. These improvements, although they would decrease the volume of CSO for high-intensity storm events usually occurring during the summer months to some extent would not adequately reduce the pollutant loads so generated to acceptable levels for discharge to the Genesee River. The proposed tunnel systems for the City of Rochester would address these higher pollutant loads to meet water quality standards for the Genseee River (2).

The improvements outlined in this section are also compatible and complimentary with the proposed tunnel system. Wastewater volumes kept in the surface conveyance system do not require pumping to the treatment facilities. In addition, the use of the tunnel system would be minimized by maximizing the use of the improved surface collection system. The tunnel system would, therefore, be operated for high-intensity storm events, thereby eliminating the frequent operation of the system which would result in maintenance costs.

IMPACT ON VANLARE TREATMENT PLANT

Concept

To realize the reduction in CSO's as a result of the implementation of the identified BMP collection system management options necessitates maximizing the capacity of the existing treatment plant. The present process capacity of the VanLare Treatment Plant is 100 MGD. Flows in excess of 100 MGD create upsets in the biological processes. A method of adequately treating increased flows to the treatment plant during wet-weather events, termed split-flow, was developed under the BMP program.

In establishing the applicability of split-flow mode of operation, several treatment plant parameters were evaluated. First, to assess the ability of the existing facilities to adequately handle increased hydraulic loads generated during storm events, a determination of the maximum hydraulic capacity of the existing treatment plant was made. Second, a determination of any process limitations that may exist during increased loading rates was made. The plant effluent must meet federal and state effluent standards regardless of the influent flowrate. Third, to manage the treatment plant more efficiently and economically during both dry- and wet-weather periods, an analysis of plant operation and performance was conducted under the application of a split-flow mode of operation.

The present process capacity of the VanLare Treatment Plant is 100 MGD with a hydraulic capacity of 200 MGD. Review of plant performance data indicated that the biological components of the treatment facility were severely upset when inflow to the plant exceeded 100 MGD. The split-flow mode of operation involved passing a maximum flowrate of 100 MGD through the biological process components while all of the inflow received full primary settling. Overall plant performance, in terms of effluent quality, was realized when the plant was operated under the split-flow mode.

Evaluation

Analyses of the treatment plant performance data were conducted to develop the process models necessary to evaluate various modes of operation of the existing facilities. In particular, process models were developed to predict the response of the primary settling units, biological clarifiers, and an optimal utilization of both primary settling and biological facilities employed under a split-flow mode of operation.

The configured models were primarily developed through the application of statistical regression techniques using 1977 plant operating data. The process models and modeling assumptions are presented in Table 51. The conventional biological BOD removal efficiency model was developed based on the response of the existing system to changes in the influent wastewater composition, wastewater flows, and mixed liquor composition.

The results of the modeling efforts are presented in the form of Figure 65.

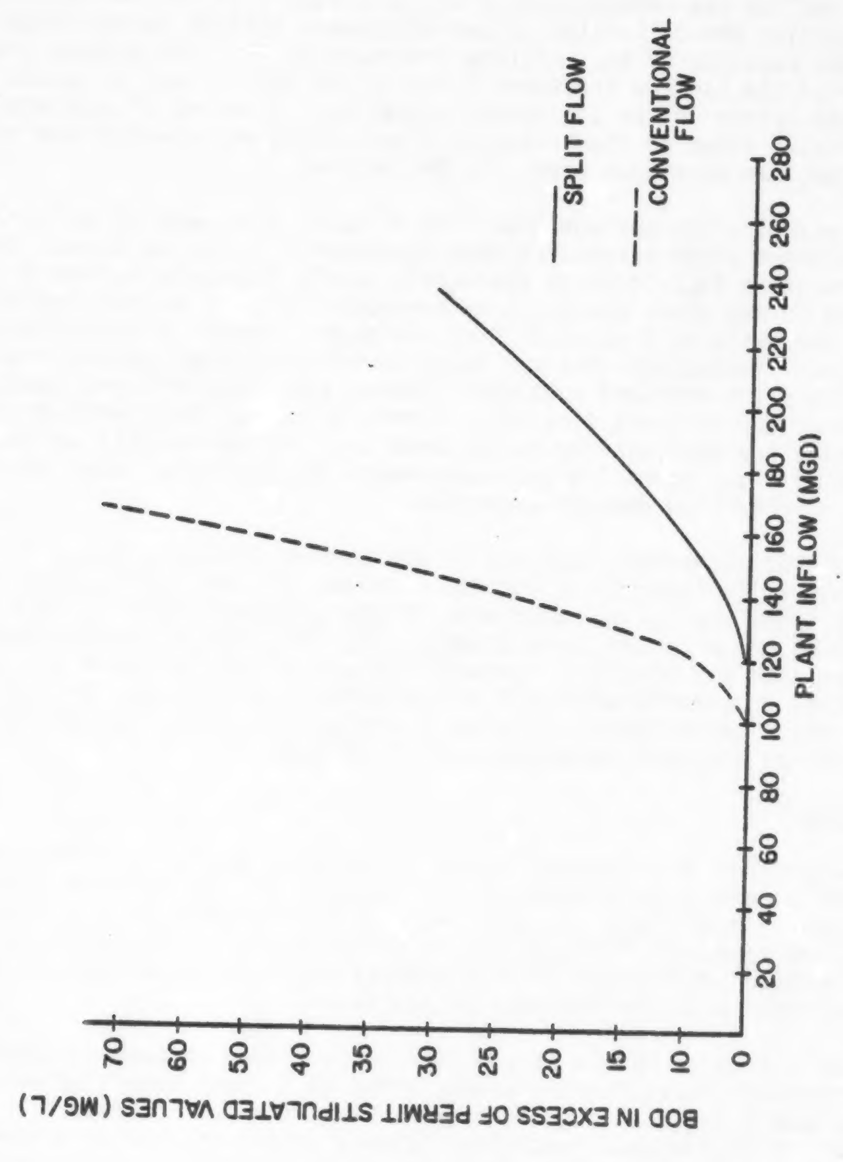


Figure 65. Model projected VanLare plant effluent quality for varying hydraulic loadings under split-flow mode of operation.

TABLE 51. PROCESS MODELS AND MODELING ASSUMPTIONS

Conventional Primary BOD Removal Efficiency

$$= 79.6\ (0.82)\ e^{-(OR/2780)}$$

Conventional biological BOD Removal Efficiency

$$= 449.3 + 0.028\ (BODi) + 0.034\ (TSSi) - 0.28\ (TKNi) - 180\ (\text{Log } Q) + 0.008\ (MLSS) - 0.016\ (MLVSS)$$

Split-Flow BOD Removal Efficiency

$$= \frac{(82\ \text{MGD})(98\%) + (Q-82\ \text{MGD})(\%\ \text{BOD Removal at } Q)}{Q}$$

BODi (influent BOD)	140.36	Concentrations reflected statistically determined average values.
TKNi (influent TKN)	15.11	
TSSi (influent TSS)	147.39	
MLSS (mixed liquor suspended solids)	1993.39	
MLVSS (mixed liquor volatile suspended solids	1565.17	
Total Surface Area of Primary Settling Tanks		149,365 ft^2
Overflow Rate (OR) - gpm/ft^2		

An analysis of the split-flow mode of operation indicated that there was an optimal method of utilizing the existing facilities that conforms to the concept of Best Management Practices. It resulted in the highest quality effluent and alleviated the problems of extended biological process degradation resulting from high-flow washout conditions. Typically, such washout conditions had resulted in a two day period of degraded performance.

The split-flow mode of operation had a number of significant advantages relative to plant operation. These are summarized as follows:

- Overall increase in annual plant performance
- Significant increase in plant performance at higher flowrates
- Savings in operating costs of approximately \$900 - \$1,000/day
- Reduction in sludge age with resultant better dewaterability
- Higher ratio of settling to biological sludge with better dewaterability
- Cost-effective plant operation.

A test program was developed to evaluate the split-flow mode of operation. An outline of the initial test program is presented as Table 52. A summary of the evaluation program is presented as Table 53.

TABLE 52. PRELIMINARY TESTING PROGRAM SPLIT-FLOW MODE OF OPERATION

Week	Split-Flow (Primary Settling to Biological
1	1.10 (5%)
2	1.20 (10%)
3	1.30 (15%)
4	1.40 (20%)
5	1.50 (25%)
6	1.60 (30%)
7	1.70 (35%)
8	1.80 (40%)
9	1.90 (45%)
10	2.00 (50%)

Note: Assumed average influent flow of 70 mgd. Measurements included TSS and BOD for plant influent, primary settling influent, primary settling effluent, and biological process effluent. On the portion of the flow that was split, similar parameters were measured on composite samples taken from the same location at two hour intervals.

TABLE 53. EVALUATION PROGRAM SPLIT-FLOW MODE OF OPERATION

1. Flow Measurement

   Primary settling influent - acquired via existing meter readings
   Primary settling treated split-flow-acquired via velocity and head measurement at point of confluence of the biological process facility bypass and the biological clarifier effluent launders. This method of flow measurement involved cross-sectional area and velocity determinations.
   Biological process treated flow - to be obtained from the above two measurements and available recycle data.

2. Frequency of Flow Measurement

   The bypass gates requiring adjustment will be set at the beginning of each week and the flow determined on a daily basis.

3. Frequency of Sample Collection

   A minimum of 4 times daily at the hours of 12 AM, 6 AM, 12 PM and 6 PM. Composite samples were collected of plant influent, primary settling influent, primary settling effluent, biological process effluent, and the chlorine contact chamber effluent.

(continued)

TABLE 53 (continued)

4. Analytical Requirements

   As outlined in Table 52.

5. Method of Providing Split-Flow

   Adjustment of the basin by-pass gates achieved the optimum process flow distribution to the biological processes in keeping with the desired daily average primary settling to biological process flow ratio.

6. Dry-Weather/Wet-Weather Operation

   The valve settings were set based on daily average flow and were not adjusted during the processing day. Diurnal variations in plant inflow were minimized by pumping from the Cross-Irondequoit Pump Station. The valve settings were maintained during wet-weather flows in order to evaluate performance under a range of hydraulic conditions.

7. Data Logging

   A log with the following minimal information was maintained:

   1. Date and time of flow measurement
   2. Influent plant flow
   3. Primary settling influent
   4. Split-flow ratio
   5. Reference settings of the necessary bypass valves
   6. Head measurement and velocity measurement in biological process effluent launder
   7. Notation as to whether sample was taken

Analysis

The data compiled on the split-flow demonstration program over the period of 8/28/78 to 10/4/78 was analyzed with respect to effluent total suspended solids concentrations and percentage reductions of BOD. The data shown plotted on Figures 66 and 67 indicate a fairly strong relationship between effluent parameters and the percentage of split-flow under dry-weather conditions. The projection of data to the 30 mg/l effluent TSS level indicates that a 25-30% split-flow under dry-weather conditions may be justified.

An analysis of the corresponding BOD data indicates a 20-25% decrease in BOD reduction under a 25-30% split-flow hydraulic regime. Phosphorus data were analyzed in a similar manner.

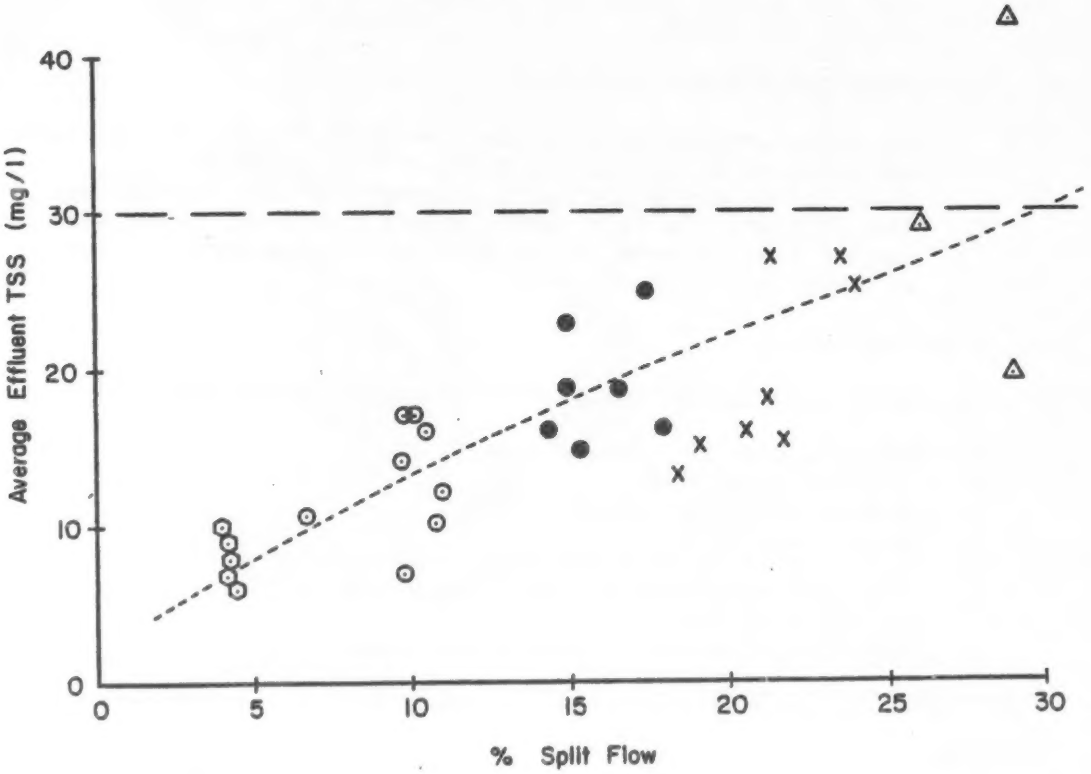


Figure 66. Split-flow TSS performance data under dry-weather flow conditions.

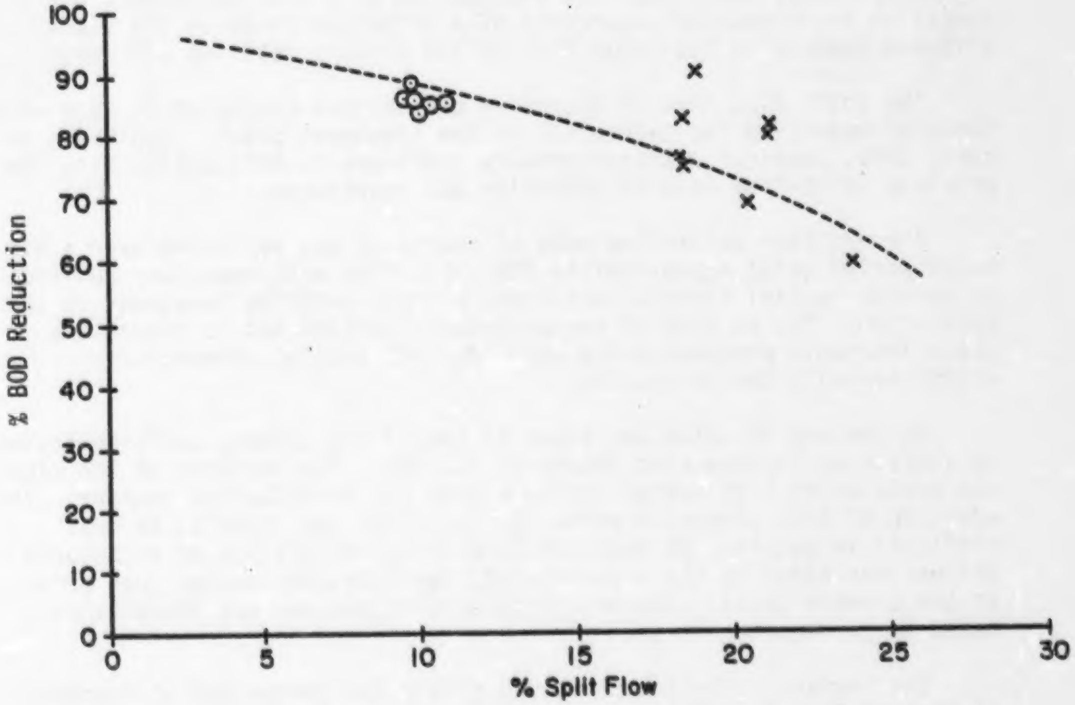


Figure 67. Split-flow BOD performance data under dry-weather flow conditions.

A problem encountered during the split-flow evaluations was the lack of rainfall necessary to generate higher plant flows. Since the percentage of split-flow decreased significantly under higher hydraulic loadings, attempts were made to adjust the biological process bypass gates immediately prior to rainfall in an effort to evaluate split-flow percentages under wet-weather conditions.

Only one day of high hydraulic loading was experienced during the test period. On September 12 an average daily flow of 114 MGD was recorded with a peak flow of 134.1 MGD measured at 12 PM. The percentage of split-flow during the test period was 5.9% and, as such, was too low to evaluate the effectiveness of the split-flow mode of operation. The collected data, however, indicated the need for a split-flow method of operation. Figure 68 shows that the relative quality of primary settling effluent increased as the flow increased. In contrast, the quality of the biological process effluent decreased with the passage of biological solids associated with higher hydraulic loadings. The application of a split-flow mode of operation prevented the occurrence of a situation in which the plant effluent quality is less than that of the primary settling effluent.

The split-flow mode of operation evaluations conducted in 1978 were based on modifying the hydraulics at the treatment plant. Beginning in early 1979, chemical assisted primary treatment in conjunction with the previous split-flow mode of operation was introduced.

The modified split-flow mode of operation was evaluated over a six month period using approximately 20% split-flow with coagulant addition to provide partial chemical-assisted, primary settling treatment to the split-flow. The purpose of the coagulant addition was to reduce the total inorganic phosphorus and colloidal BOD and TSS concentrations associated with the split-flow.

On January 18, alum was added to one of the primary settling basins to achieve an average alum dosage of 100 ppm. The addition of the alum was assisted by high-energy mixing within the distribution launder. The addition of alum created a pinpoint floc which was found to be most difficult to settle. On February 8, a dosage of 0.5 ppm of an anionic polymer was added to the alum-treated, degritted wastewater just prior to the primary basin. The dosage of anionic polymer was established based on jar tests.

The average operating performance data for the period of February 11 through June 3, 1979 are presented as Table 54. The weekly average daily flow during the evaluation period was 106.2 MGD with an average BOD, TSS and TIP effluent concentrations of 28.7 mg/l, 25.7 mg/l, and 0.86 mg/l, respectively. These performance data were obtained under an average split-flow percentage of 25.8%. The detailed performance data are presented in the form of Figures 69 and 70.

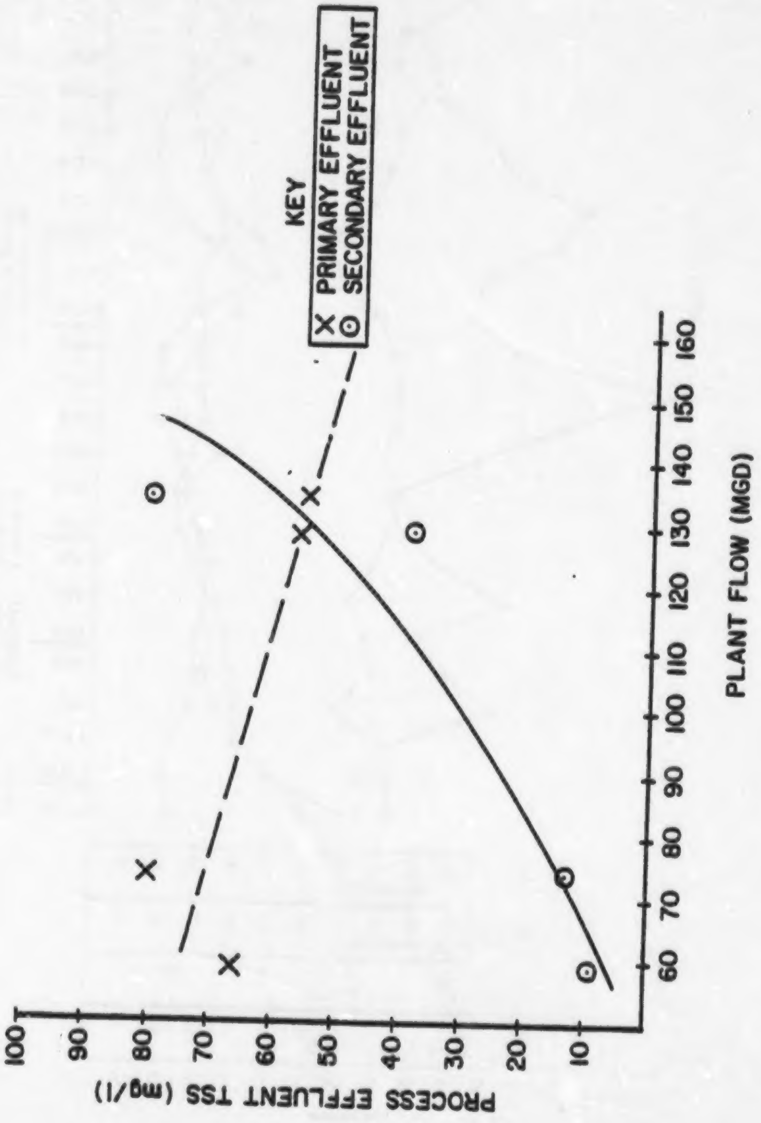


Figure 68. VanLare process performance data.

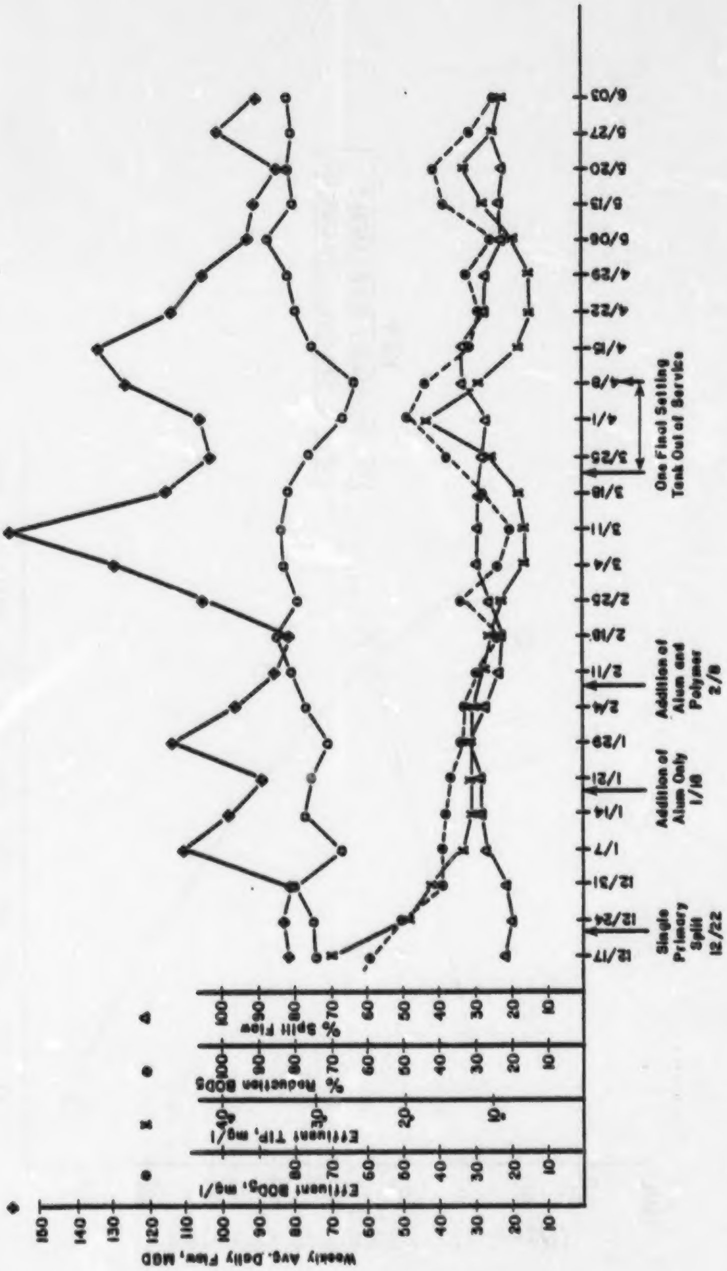


Figure 69. BOD and TIP effluent concentrations under chemically assisted split-flow mode of operation.

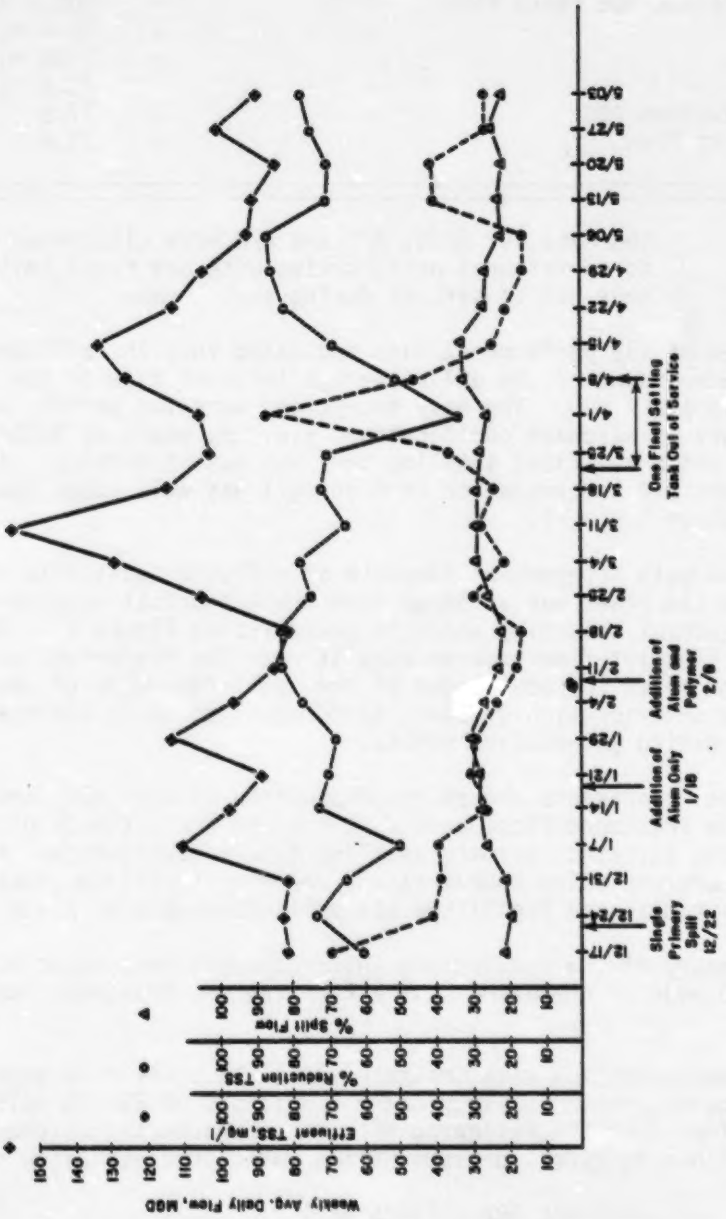


Figure 70. TSS effluent concentrations under chemically assisted split-flow mode of operations.

TABLE 54. SPLIT-FLOW ANALYSIS OPERATING PERFORMANCE DATA 2/11/79 - 6/03/79*

Weekly Average Daily Flow	- 106.2 MGD
BOD	- 28.7 mg/l
TIP	- 0.86 mg/l
TSS	- 25.7 mg/l
% Reduction TSS	- 77.5
% Split Flow	- 25.8

Note: The data for 3/25, 4/1 and 4/8 were eliminated due to operational difficulties with one final settling tank out of service during these weeks.

Review of the performance data indicated that the effluent TSS was largely independent of the daily average influent flow to the plant, even at flows up to 158.7 MGD. The only exceptions were two periods which were controlled more by hardware modification, i.e. the weeks of 3/25/79, 4/1/79 and 4/8/79 in which one final settling tank was out of service. In addition, the effluent TIP concentration of 0.86 mg/l was well under the permit stipulated level of 1.0 mg/l.

The largely independent response of effluent quality to the hydraulic loading to the plant was a change from the historical dependence of the plant on hydraulic loading which is presented as Figure 71. Comparison of the Phase II split-flow program results with the historical performance clearly indicated the advantages of the split-flow mode of operation in reducing the deterioration of plant performance at daily average flowrates encountered during wet-weather events.

Figure 72 presents the point of addition of both alum and polymer to the VanLare Treatment Plant process train and the ultimate disposition of the chemical assisted, primary settling treated split-flow. Also shown on Figure 72 are the valve operators and metering facilities that were necessary to optimize and facilitate the split-flow mode of plant operation.

A summary of the conclusions and recommendations associated with the split-flow mode of operation at the F.E. VanLare Treatment Facility is as follows:

1. The demonstration data indicated that the split-flow mode of operation of the treatment facility under conditions of 25-30% split-flow allowed attainment of the following effluent characteristics under daily average flows in excess of 150% of the rated process design capacity:
    - effluent BOD 20-25 mg/l
    - effluent TIP 0.75-1.00 mg/l

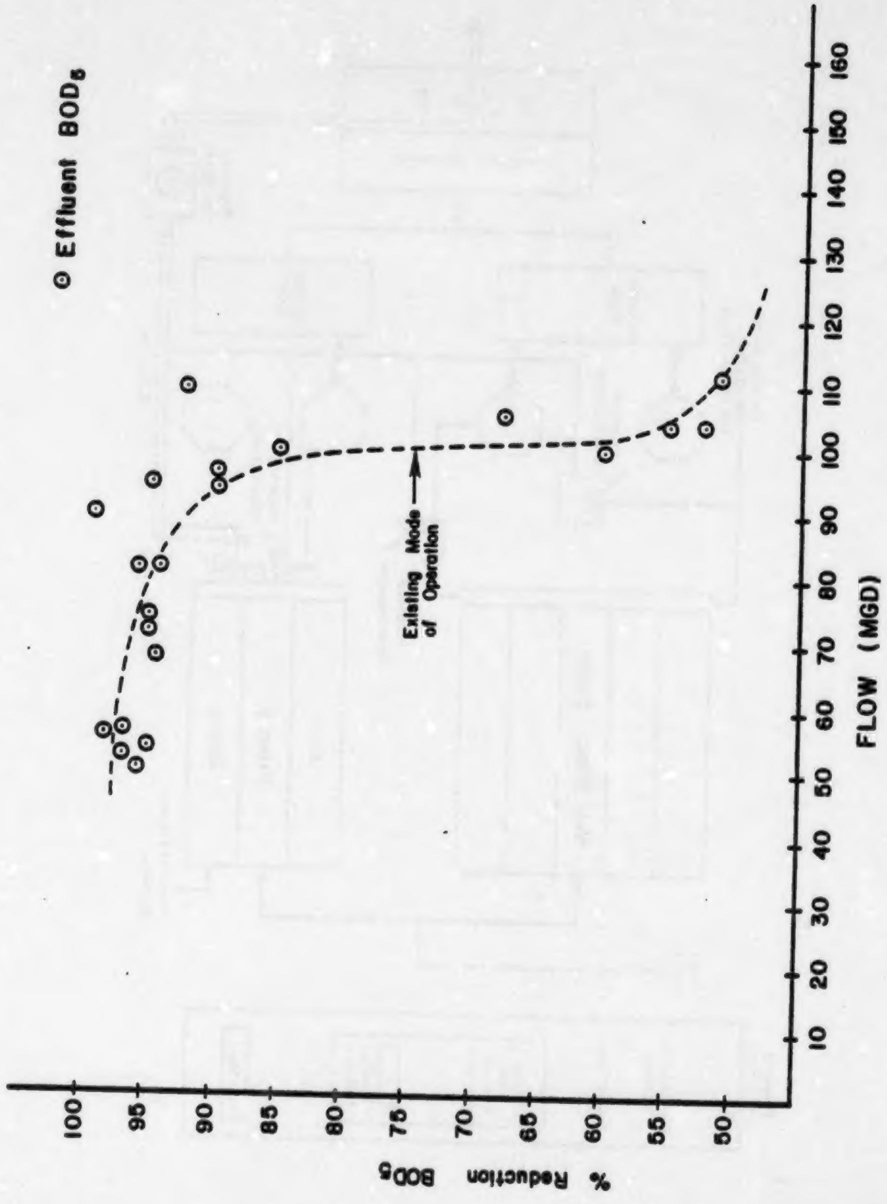


Figure 71. Historical dependence of VanLare plant effluent quality on hydraulic loading.

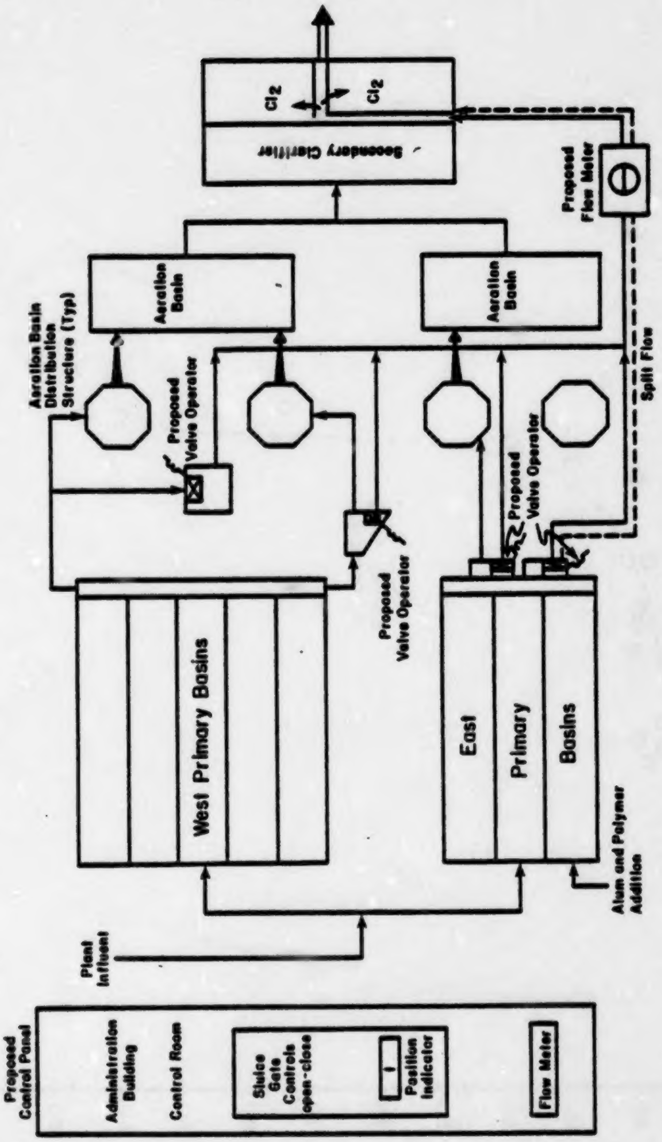

Proposed Control Panel
Administration Building
Control Room
Sluice Gate Controls open-close
Position Indicator
Flow Meter
Plant Influent
West Primary Basins
East Primary Basins
Alum and Polymer Addition
Proposed Valve Operator
Proposed Valve Operator
Proposed Valve Operator
Aeration Basin Distribution Structure (Typ)
Aeration Basin
Aeration Basin
Split Flow
Proposed Flow Meter
Secondary Clarifier
Cl_2
Cl_2

Figure 72. Split-flow valving and instrumentation requirements.

2. A comparison of the split-flow mode of operation to the normal mode of operation of the treatment facility indicated significant improvement in average effluent quality associated with implementation of the split-flow operating concept. The split-flow performance was based on data obtained during the spring of 1979 while the baseline period was the spring of 1978.

3. The split-flow mode of operation provided the flexibility of meeting effluent limitations under a much broader range of hydraulic loading conditions than available under the normal mode of operation.

4. The use of alum, in conjunction with an ionic polyelectrolyte at concentrations ranging from 50-100 mg/l and 0.5-1.0 mg/l, respectively, overcame the BOD and nutrient effluent quality limitations experienced under the initial demonstration work conducted during the fall of 1978.

5. To facilitate the full scale implementation of the split-flow mode of operation, facilities should be provided to store and meter alum and polymer. The point of addition should be modified so as to provide flash mixing at the point of alum addition and adequate mixing flocculation energy at the point of polymer addition.

6. Appropriate instrumentation including four valve operators, controls, and an additional flow meter needed to facilitate the effective control of the split-flow mode of operation of the treatment facility should be installed.

7. Only by operating the treatment facilities under the split-flow mode can the plant effectively treat wet-weather flows from the present collection system. Without additional stormwater treatment facilities, the present treatment facilities cannot handle the increased volume of wet-weather flows generated from the implementation of the BMP system improvements except under the split-flow mode of operation.

8. The split-flow mode of operation offers an acceptable means of decreasing the costs associated with the treatment of dry-weather flows while achieving the discharge permit stipulated effluent quality of BOD, TSS, and TIP. Furthermore, the split-flow mode of operation offers the only cost-effective method of attaining the TIP effluent standard under dry-weather conditions.

9. For treatment plants serving combined sewer systems, effluent discharge limitations should be established on the basis of pollutant mass loadings and not on a concentration basis. In any event, limitations presently based on the generally accepted standard of 30-30 and 85% removal (R) should be abolished. The 30-30 refers to 30 mg/l BOD and TSS computed as an average over any 30 consecutive day period. A limitation involving both 30-30 and 85% R assumes that the influent solids concentration is 200 mg/l. For plants receiving less than 200 mg/l TSS, to meet the 85% R limitation, implies that less than 30 mg/l BOD and TSS can be discharged. This would not be equitable to all biological process treatment plants.

SECTION 8

RECEIVING WATER STUDIES

BENTHIC DEMAND

Background

One of the major objectives of the BMP program was the identification of dissolved oxygen (DO) impacts on the Genesee River resulting from CSO's. The data gathering effort was, therefore, directed toward identifying both the immediate and transient impacts attributable to the CSO itself as well as establishing the long-term impacts associated with the build-up of benthic sludge deposits.

Dissolved oxygen impacts attributable to storm runoff pollutants have been difficult to demonstrate conclusively in any area in the country, although justification of most abatement facilities is primarily based on mitigation of such impacts. Reasons for this difficulty have been the logistical problems associated with monitoring transient, short-term storm events, the complexity of runoff-loaded river systems, and a rudimentary framework for the analysis of benthic oxygen demands. Velz, through laboratory experiments, made significant contributions in mathematically describing the long-term effects of benthic oxygen demands (31). However, his results did not fully address the issue of the original source of benthic deposit material. Methods have recently been developed for measurement of benthic demands, onsite or in the lab; however, the lack of a rigorous description of the sludge build-up process prevents water quality projections under varying natural conditions.

To build on these deficiencies, the receiving water impact investigations conducted under the BMP program involved the establishment of a continuous water quality monitor for dissolved oxygen and turbidity determinations in the water column, as well as the placement of sediment traps to enable the measurement of specific constituent concentrations and sediment oxygen demand (SOD). Locations for these monitors were selected to show the effects of CSO's in terms of water column deoxygenation from benthic sludge accumulation. Projections of receiving water DO concentrations were made using a previously developed Genesee River, steady-state DO model with updated data obtained under the BMP program.

Immediate impacts of soluble organics in CSO's were to be identified from transient DO depressions recorded during both previous water quality surveys and as shown by the recently installed water quality monitor. Impacts resulting from the settleable materials in the overflow were determined from the variations in rate of accumulation and constituent concentrations found in the sediment, as well as variations in the rate of sediment oxygen uptake.

Field Program

Measurement of sediment buildup and benthic oxygen demand was accomplished by the installation of five sediment traps along the Genesee River as shown in Figure The exact locations are identified as follows:

1. Upstream of the Elmwood Avenue bridge
2. Downstream of the Eastman Kodak treatment plant (approximately 200 ft downstream of the plant effluent discharge)
3. Boxart Street (most southerly point of commercial shipping along the Genesee River)
4. Stutson Street Bridge
5. Upstream of the Eastman Kodak Treatment Plant

These sites were selected to best indicate both the transient and long-term impacts of CSO's on receiving water quality in light of the varying conditions along the Genesee River. Location 1, immediately upstream of the Elmwood Avenue bridge, represented the portion of the River upstream from CSO discharges. At this location, the river was characterized by low velocities and the widest channel section as it passed through the City of Rochester.

Location 2 represented the first reach of the river immediately downstream of all major CSO and stormwater discharges. This site was also downstream of the three falls occurring within the City of Rochester. The river within this reach was characterized by a narrow, deep cross-section. River velocities are low, except during large storm events, causing much of the settleable material that was carried downstream through the city to settle out.

Location 3 represented the most southerly, navigable point along the Genesee River. Commercial shipping operations were conducted upstream to a point opposite Boxart Street. The river channel from the mouth at Lake Ontario to this sampling station was maintained by the Army Corps of Engineers. Annual dredging operations were conducted between mid to late summer.

The most downstream location, Site 4, represented the portion of the Genesee River characterized by very low velocities and occasional current reversals due to the influence of Lake Ontario. It also represented that portion of the river where maximum DO depressions were previously predicted and measured.

These four locations adequately represented the varying conditions along the river to properly assess the urban influence on sediment quality. However, after initial sampling, it was determined that fluctuations in the quality of the Eastman Kodak Treatment Plant discharge can disguise the effect of CSO's on SOD. Therefore, a decision was made to install a fifth sediment trap immediately upstream of the plant's discharge.

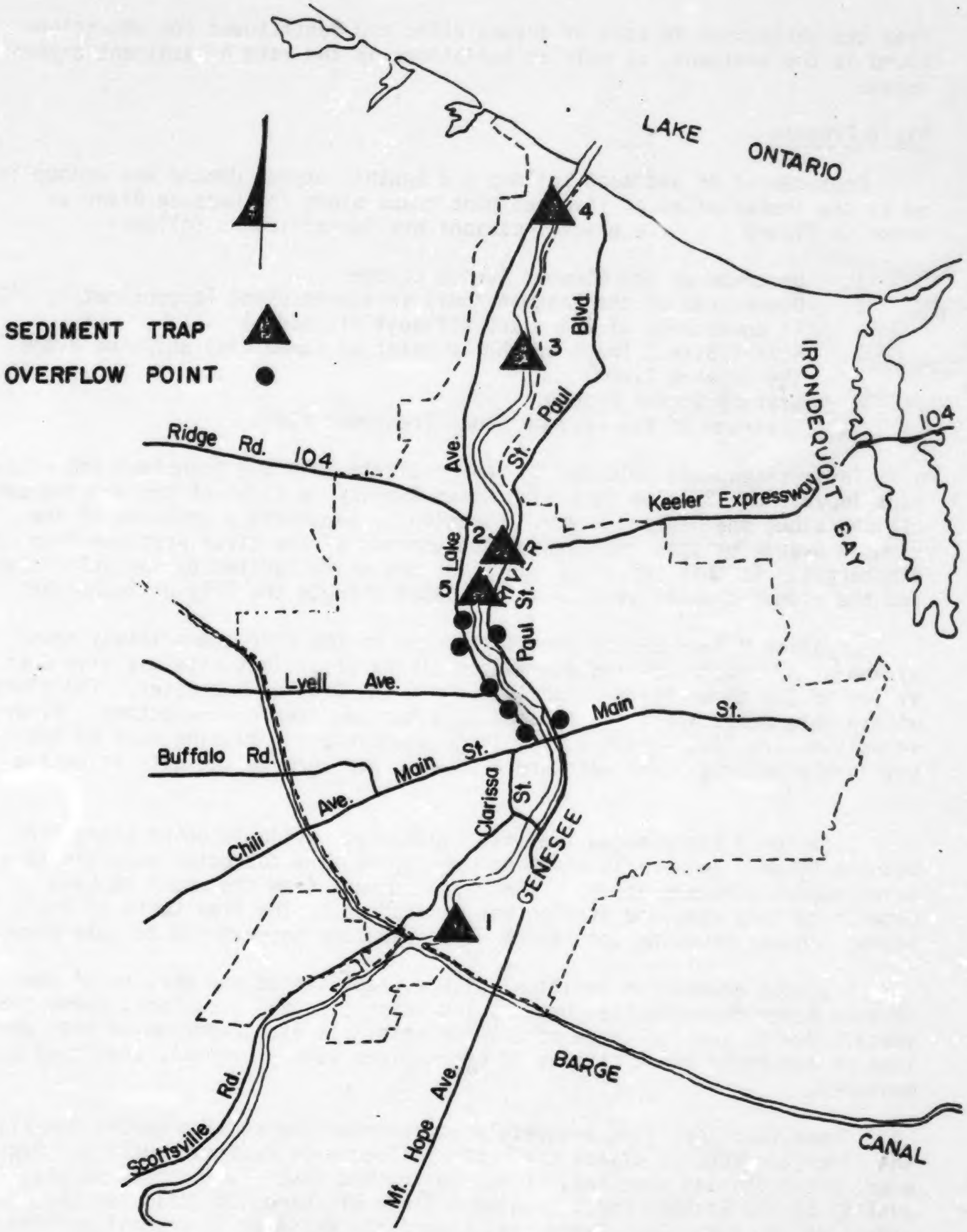

LAKE ONTARIO
IRONDEQUOIT BAY
SEDIMENT TRAP
OVERFLOW POINT
4
3
2
5
1
St. Paul Blvd.
Lake Ave.
St. Paul St.
RIVER
GENESEE
Ridge Rd. 104
104
Keeler Expressway
Lyell Ave.
Main St.
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
Scottsville Rd.
Mt. Hope Ave.
BARGE CANAL

Figure 73. Sediment trap locations.

To obtain representative data at reasonable cost and to allow for easy site access for periodic sample collection and maintenance, a simplified approach was adopted and implemented. Sediment traps constructed of plexiglass and provided with heavy weights as anchors were lowered into the river at the five identified locations and allowed to rest at the bottom. The location of each trap was marked with the use of a buoy attached to the unit. Figure 74 shows the general configuration of each sediment trap. Sampler inspection and sediment collection were accomplished by use of a winch mounted on the back of a small boat. Figure 75 shows the operation of removing a trap for sediment measurement.

An evaluation plan was developed and an optimum data collection schedule adopted. Each trap was attended to on a biweekly basis, except that after large storm events, traps were inspected to insure that they were not washed away or had tipped over. Table 55 presents the evaluation plan for the benthic demand studies.

TABLE 55. BENTHIC DEMAND STUDIES EVALUATION PLAN

Week	Sedimentation Rate Determination	Maintenance	Sediment Analysis	Benthic Oxygen Demand
1	X			
2	X	X	TSS, VSS	
3	X			
4	X	X	TSS, VSS	
5	X			
6	X		TSS, VSS, Pb Zn, Cd, Hg	X
7	X			
8	X	X	TSS, VSS, Pb Zn, Cd, Hg	X
9	X			
10	X	X	TSS, VSS, Pb Zn, Cd, Hg	X
11	X			
12	X	X	TSS, VSS, Pb Zn, Cd, Hg	X

Method of Analysis

To determine the rate at which sediment accumulated in the traps, a simplified approach was adopted. Every two weeks when the traps were inspected a sediment depth measurement was made. Since the material collected seldom was uniformly deposited over the bottom of the trap, depths were taken at various points over the trap bottom and the results averaged.

After the depth measurements were recorded, most of the clear water in the trap was then poured out until a known volume of sediment and water re-

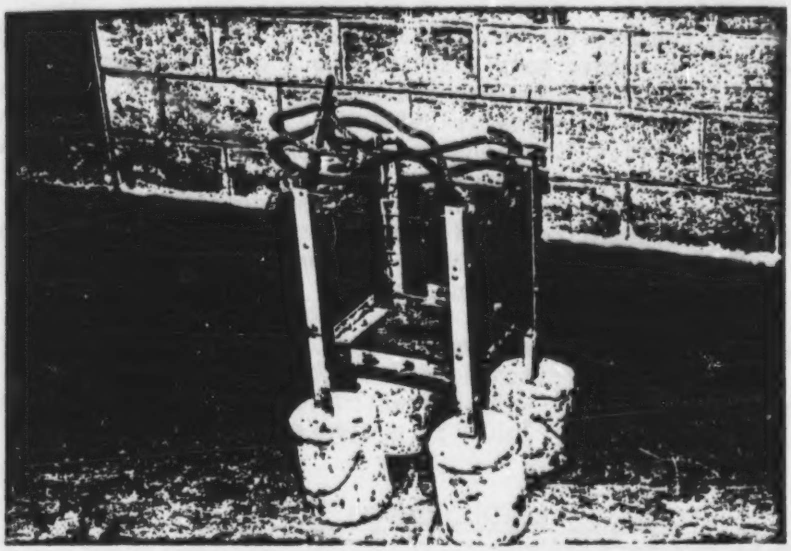


Figure 74. Sediment trap prior to installation.

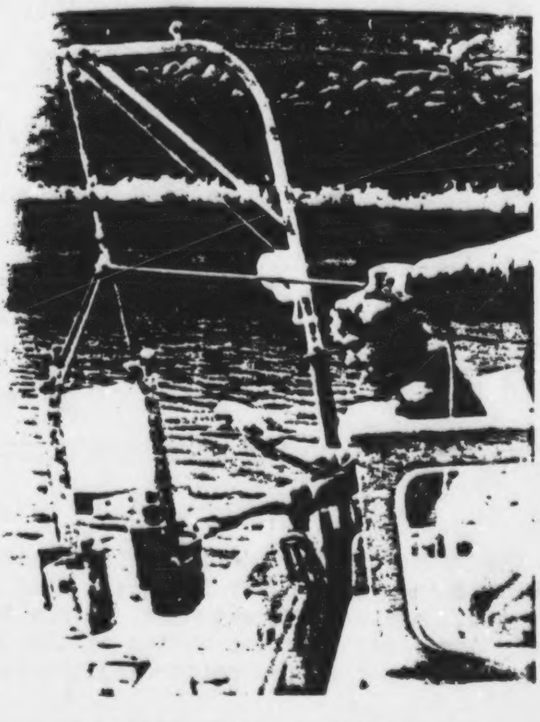


Figure 75. Removal and inspection of sediment trap.

mained. This volume was then mixed thoroughly until no visible sediment remained on the bottom. From this mixture, a sample was collected and taken to the laboratory for the required analyses.

SOD determination was made by establishing an oxygen depletion curve in the laboratory for collected trap sediment. The collection process involved obtaining a quantity of accumulated trap sediment prior to mixing of trap water required for other analytical analyses. This procedure was considered adequate, since much of the benthic oxygen demand is exerted by the top layer of sediment.

To delineate the impact of CSO's on sediment quality from those impacts induced by the Kodak Treatment Plant discharges, a fifth sediment trap, identified as Site 5, was installed in 1979. The Army Corps of Engineers' summer dredging operations along the Genesee River from the Boxart location (Site 3) to the mouth of the river created some initial installation and maintenance problems but these were subsequently resolved.

Results

The sediment trap data collected under this program are presented in Table 56. The data include the observed sediment depth along with the total and volatile solids for each of the sampling dates. Both adverse weather and occasional river conditions precluded data collection from the 12 storm events set forth in the implementation schedule. The dry weight sediment concentrations for lead, cadmium, zinc and mercury are also shown. It is interesting to note that the Elmwood sampling site (Site 1) had no measurable cadmium and the lowest measured lead concentrations. In contrast, the first monitoring site downstream of CSO discharges (Site 2) has the highest measured lead concentrations and significant cadmium concentrations.

Heavy metals were used as tracers, that is, the presence of heavy metals in the river would be indicative of similar parameters associated with urban stormwater runoff.

The profile of the heavy metal sediment concentrations is better understood by analyzing the data presented on Figure 76. Figure 76 presents the sediment concentrations for lead (Pb), cadmium (Cd) and mercury (Hg) and the percent volatile solids as a function of distance from the mouth of the Genesee River. The portion of the river between mileposts 5 and 10 received CSO discharges. This same portion of the river was also characterized by high velocities. In contrast, the portion of the river between mileposts 0 and 5 was characterized as quiescent and, therefore, condusive for sediment deposition.

Figure 76 illustrates an example of a river with uncontaminated sediments upstream of an urban area. Both stormwater and CSO's discharge to the river during periods of wet-weather as it passes through the urban area. The heavy metals in these discharges are predominantly in a particulate form which settle in the lower reaches of the river. Because of this characteristic, an exponential fall-off of the sediment metal concentrations as a

TABLE 56. SEDIMENT TRAP ANALYSIS

Site No.	Sampling Date	Sampling Period Days	Sediment Depth-In.	TS g	VS g	mg/kg dry wt.			
						Pb	Cd	Zn	Hg
Elmwood 1	6/26	13	0.13	96	9	48.0	0.0	275	2.00
	7/11	15	0.13	19	2	0.0	0.0	510	0.0
	7/24	13	0.13	24	2	38.8	0.0	589	1.55
	8/08	15	N/A	39	4	50.0	0.0	250	2.60
	8/21	13	0.13	46	3	32.1	0.0	289	0.80
	9/11	21	0.25	94	8	42.8	0.0	276	0.80
	9/25	13	0.75	963	59	N/A	N/A	N/A	N/A
	10/10	13	0.13	266	19	N/A	N/A	N/A	N/A
	10/23	12	0.75	N/A	N/A	N/A	N/A	N/A	N/A
Kodak (down) 2	6/26	15	0.25	26	3	94.0	0.0	594	1.67
	7/12	16	0.25	107	9	97.3	14.7	349	0.00
	7/24	12	0.13	31	3	176.0	14.2	690	0.00
	8/08	15	0.38	69	6	143.7	20.8	633	0.00
	8/22	14	0.25	65	6	82.6	17.0	448	0.70
	9/11	20	0.50	323	26	119.0	8.4	238	0.00
	9/25	13	4.00	1377	233	N/A	N/A	N/A	N/A
	10/10	13	0.25	263	22	N/A	N/A	N/A	N/A
	10/10	12	1.00	N/A	N/A	N/A	N/A	N/A	N/A
Boxart	6/26	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	7/12	16	0.75	910	48	13.4	2.9	49	0.05
	7/24	12	0.13	20	2	61.5	0.0	661	2.16
	8/08	15	0.75	153	9	112.0	18.7	503	0.70
	8/22	14	0.50	167	14	116.0	18.6	416	0.20
	9/11	20	0.38	87	7	108.0	19.7	444	0.70
	9/25	13	0.38	146	13	N/A	N/A	N/A	N/A
	10/10	13	0.25	526	37	N/A	N/A	N/A	N/A
	10/23	12	0.75	N/A	N/A	N/A	N/A	N/A	N/A

(continued)

TABLE 56 (continued)

Site No.	Sampling Date	Sampling Period Days	Sediment Depth-In.	TS g	VS g	mg/kg dry wt. Pb	Cd	Zn	Hg
Stutson 4	6/26	15	8.00	4300	196	40.7	6.6	193	0.12
	7/12	16	3.00	2900	136	27.0	4.8	112	0.10
	7/24	12	0.38	281	15	69.7	10.6	266	0.02
	8/08	15	1.00	184	10	70.7	8.5	341	0.46
	8/22	14	0.50	433	28	48.0	14.1	176	0.00
	9/11	20	0.25	336	25	105.0	14.4	340	0.30
	9/25	13	0.75	262	20	N/A	N/A	N/A	N/A
	10/10	13	0.25	307	25	N/A	N/A	N/A	N/A
	10/23 (trap lost)	12	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Kodak (up)	6/26	15	0.13	59	6	143.0	16.1	448	0.48
	7/12	16	0.25	139	13	204.0	8.2	400	0.41
	7/24	12	0.13	40	4	316.0	0.0	775	1.22
	8/08	15	0.25	336	29	228.0	15.4	716	0.40
	8/22	14	0.25	150	17	116.0	14.1	390	0.00
	9/11	20	0.25	291	26	208.0	8.3	387	0.60
	9/25	13	2.00	748	75	N/A	N/A	N/A	N/A
	10/10	13	0.25	353	28	N/A	N/A	N/A	N/A
	10/23	12	1.00	N/A	N/A	N/A	N/A	N/A	N/A

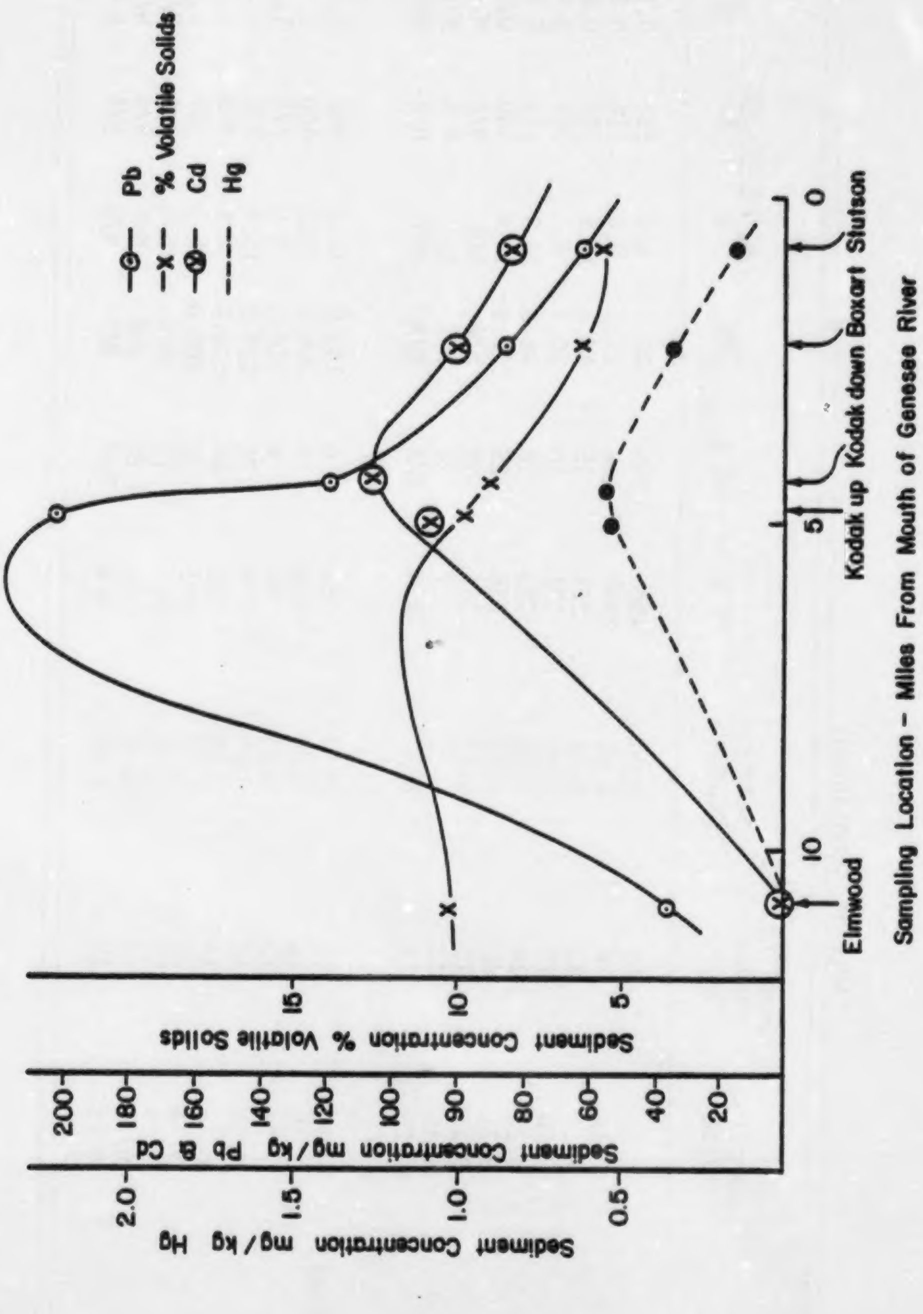


Figure 76. River profiles of heavy metal sediment concentrations.

function of river mileage occurs resulting in the lowest heavy metal sediment concentrations being observed at the Stutson Street location.

The data in Figure 76 imply that the CSO's and stormwater discharges are responsible for the heavy metal concentrations in the lower reaches of the Genesee River. This is further supported by the analysis presented in Figure 77, which presents the sediment lead concentrations at the Kodak upstream site as a function the number of days having greater than 0.25 in. of precipitation. The sediment lead concentrations correlated reasonably well to total precipitation measured during the sampling period. However, the latter part of August and the first part of September are generally characterized by a large number of low-intensity rainfall events, which contribute little to runoff and therefore little to CSO's. An improved level of correlation was, therefore, observed when a comparison was made between the total number of days having greater than 0.25 in. of precipitation against the lead concentrations measured at the Kodak upstream site.

It was found that the uncontaminated sediment, represented by the Elmwood site, urban runoff contaminated sediment, represented by the Kodak upstream site, and CSO generated solids exhibited volatile solid concentration ratios which were consistent with expected results. These ratios are presented as follows:

$$(\mathrm{Pb}/_{\mathrm{TVS}})_{\mathrm{CSO}} = 4.23\times10^{-4}$$

$$(\mathrm{Pb}/_{\mathrm{TVS}})_{\substack{\mathrm{CONTAMINATED}\\ \mathrm{SEDIMENT}}} = 2.05\times10^{-4}$$

$$(\mathrm{Pb}/_{\mathrm{TVS}})_{\substack{\mathrm{UNCONTAMINATED}\\ \mathrm{SEDIMENT}}} = 2.62\times10^{-5}$$

Figure 78 presents the average and range of the sedimentation rate data for each of the Genesee River sediment monitoring sites. Table 57 presents the laboratory established sediment oxygen uptake in g $O_2/m^2/d$.

TABLE 57. OXYGEN UPTAKE OF BOTTOM SEDIMENTS IN THE LABORATORY

Location	Site	Oxygen Consumption g $O_2/m^2/d$
Elmwood	1	0.15
Kodak (upstream)	5	0.08
Kodak (downstream)	2	0.23
Boxart	3	0.05
Stutson	4	0.09

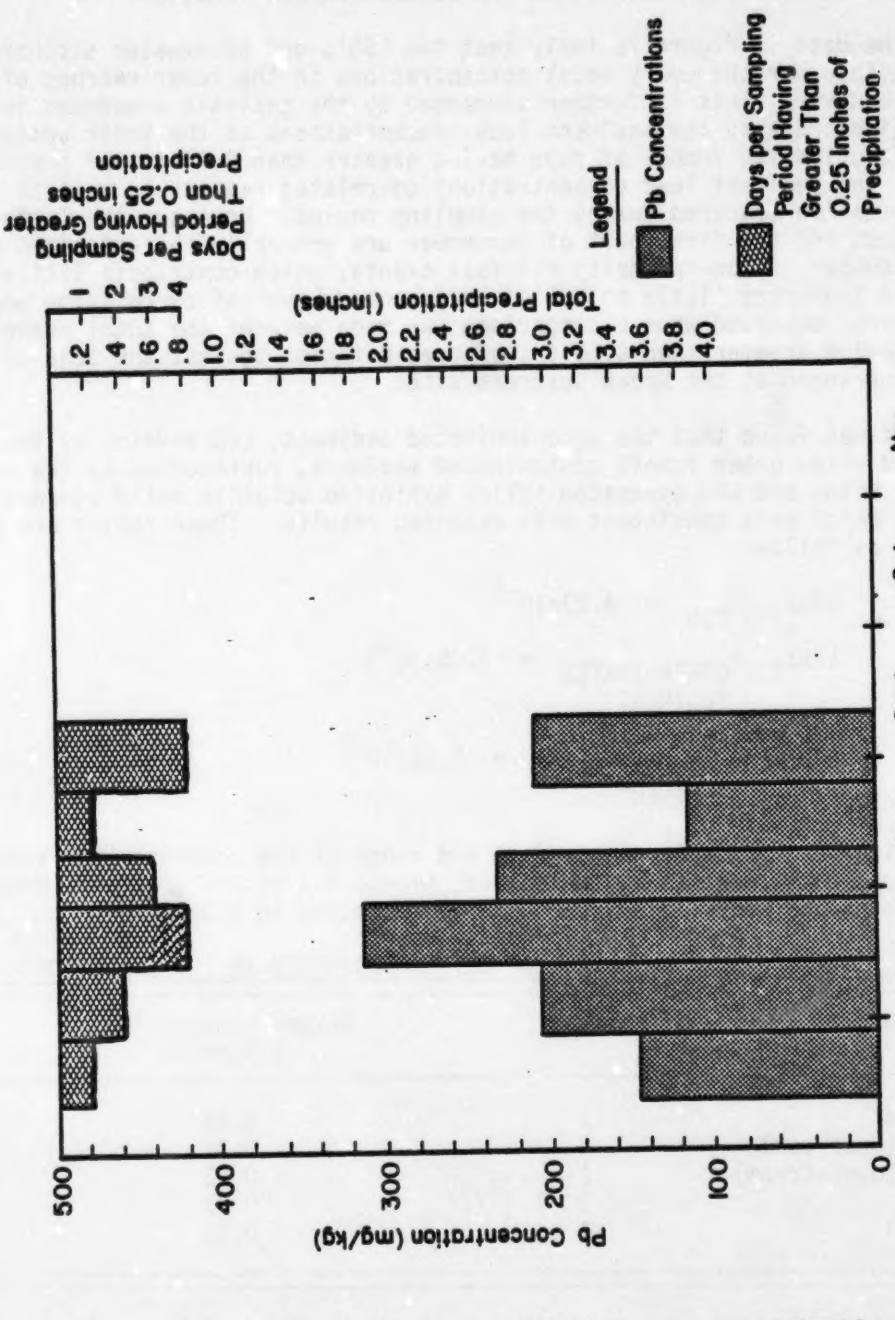


Figure 77. Sediment lead concentrations vs. precipitation.

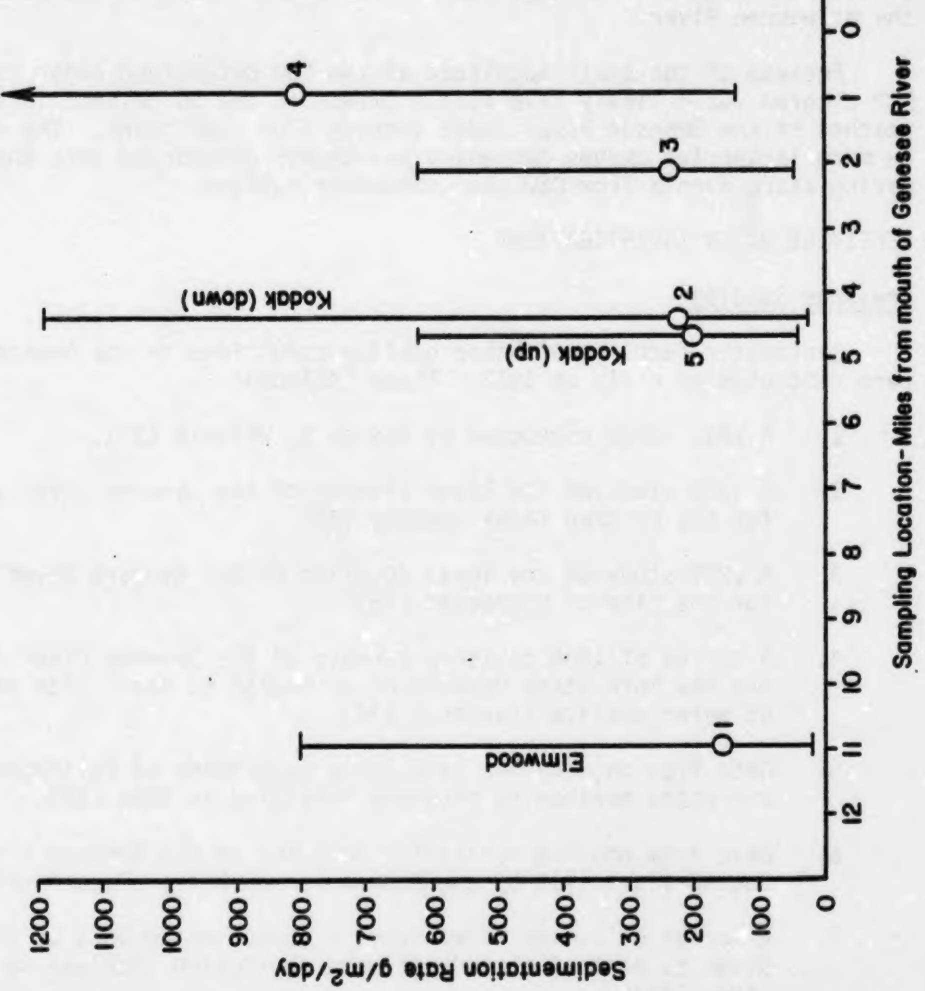


Figure 78. Sedimentation rate data for sediment monitoring sites.

Sediment oxygen demand (SOD) tests were conducted on the Milwaukee River during a recent CSO study (32). The results indicated that a significant difference between SOD rates for disturbed and undisturbed conditions, such as these conducted under bench scale testing, can exist. In that study, disturbed SOD readings exceeded undisturbed laboratory readings by as much as a factor of 1000. SOD values of nearly 1400 g $O_2/m^2/d$ resulted from scouring of bottom sediments which occurred from submerged CSO's. The report concluded that high SOD readings can have a severe impact on the DO balance of the Milwaukee River.

Because of the small magnitude of the SOD determined under the Rochester BMP program would likely have little effect on the DO balance in the lower reaches of the Genesee River under average flow conditions. The effect would be much larger for oxygen demanding pollutants discharged into the River during storm events from CSO and stormwater outlets.

RECEIVING WATER INVESTIGATIONS

Previous Studies

Systematic sampling of water quality conditions in the Genesee River were conducted as early as 1912. These included:

1. A 1912 study conducted by George C. Whipple (33).
2. A 1929 study of the lower five mi of the Genesee River prepared for the Eastman Kodak Company (34).
3. A 1929 study of the lower 10 miles of the Genesee River prepared for the City of Rochester (35).
4. A series of 1954 sanitary surveys of the Genesee River conducted by the New York State Department of Health to facilitate establishment of water quality standards (36).
5. Data from ongoing New York State Department of Environmental conservation monitoring programs initiated in 1966 (37).
6. Data from ongoing monitoring programs on the Genesee River conducted since 1965 by the Monroe County Health Department (38).
7. A series of summer river surveys conducted in 1973 on the Genesee River as part of a study of water pollution problems in the Great Lakes (39).

Although the historical data satisfactorily documented past water quality in the Genesee River, it was not adequate with respect to spatial detail. Most of the surveys lacked CSO and stormwater discharge information required to properly assess water quality. Therefore, four extensive water quality surveys of the Genesee River were performed during 1975, two of which were conducted during storm events (40). The results from these and earlier river surveys were used to develop, calibrate, and verify a water quality model of the Genesee River.

In general, Genesee River water quality data obtained since 1973 indicated that water quality was not generally stressed during dry-weather flows. The level of carbonaceous BOD was less than 10 mg/l and the nitrogenous demand was usually less than 5 mg/l. Contravention of the DO standard of 5.0 mg/l occasinally occurred during dry-weather, during low river flow and high ambient temperature conditions. The DO standard under these conditions was marginally violated in the lower reaches of the river. Fecal coliform concentrations during dry-weather were generally not in violation of standards, although some data indicated that severe short-term violations occurred as a result of many point source discharges in the basin.

During wet-weather events the Genesee River experienced measurable water quality degradation. Carbonaceous BOD concentrations above 40 mg/l were observed and nitrogenous BOD often exceeded 25 mg/l. The resulting DO concentrations were below 2 mg/l. Fecal coliform concentrations were also high and often exceeded 100,000 cells/100 ml. This contravention of standards was though to result from stormwater and CSO discharges to the river.

General

The intent of the receiving water studies conducted as part of the overall Rochester BMP program was three-fold:

- To determine the present impact of CSO discharges on the quality of the Genesee River and Lake Ontario
- To provide a continuous assessment of implemented control strategies as developed under the BMP program
- To determine the improvement of receiving water quality upon implementation of potential structurally-intensive abatement alternatives.

The resulting product of the three phase program was to be utilized by the Monroe County Pure Waters Agency for feedback and control of its overall operations with its ultimate objective of protecting the receiving waters.

Under the BMP program, long-term monitors and associated data analyses were intended to provide a better understanding of the nature of receiving water impacts of CSO's through a correlation of transient precipitation episodes and water quality trends with rainfall and overflow occurrences from the various drainage basins.

In addition to the long-term water quality monitoring of the Genesee River, an analysis of long-term continuous time series data of receiving water quality records was conducted. Currently, the Monroe County Department of Health obtains water samples from the Genesee River on an hourly basis during the months of May to September. An important parameter measured is fecal coliform bacteria, because of its impact on the beaches. As part of the BMP program, the data base acquired by agencies such as the Health Department was examined in an attempt to develop a relationship between fecal coliform counts and CSO discharges.

Design of Field Program

One of the primary objectives of the receiving water investigations was to establish a continuous water quality monitor on the Genesee River. The configured system was to measure the following parameters and store the results on a cassette tape for subsequent data reduction and analysis:

- Temperature
- Dissolved oxygen
- Conductivity
- Turbidity (percent light transmission)

The instrument selected for field installation was an automatic water quality analyzer, Model Mark VIII, manufactured by Martek Instruments, Inc. This system comprised a single electronic module. Internal subsystems allowed interfacing to a remote sensor package, sensor signal conditioning, cassette tape data recording, and digital displaying of parameteric data in engineering units.

Power to the system was provided by an external 12 VDC source with a trickle charger, which maintained the battery at full capacity. A self-contained power switching unit activated the sensors and signal conditioning electronics only during programmed data recording intervals. This facilitated monitoring operations during any extended periods of AC power outage.

Included in the total system package were a data reader and computer interface. The data reader provided a visual display of the recorded data from magnetic tapes and provided a suitable signal, with the aid of the interface, for an external data processor.

Site Selection--

Previous Genesee River water quality modeling investigations have indicated that maximum DO depressions as a result of CSO discharges occur in the lower reaches of the river near Lake Ontario. This reach is known for low velocities and even flow reversals due to the influence of the lake (40). Based on these conditions, the water quality monitor was installed on the Stutson Street Bridge. Figure 79 shows the location of the water quality monitoring installation.

The recording unit to the monitoring system which was housed in a steel shed located on a wooden pier below the bridge. The monitoring unit containing the various measurement probes was located in approximately 15 ft of water directly off the eastern bridge pier. AC power was available from the control house located on the bridge. The necessary power lines were installed and provided the monitoring system with full time recording capability without the constant need to recharge or replace batteries.

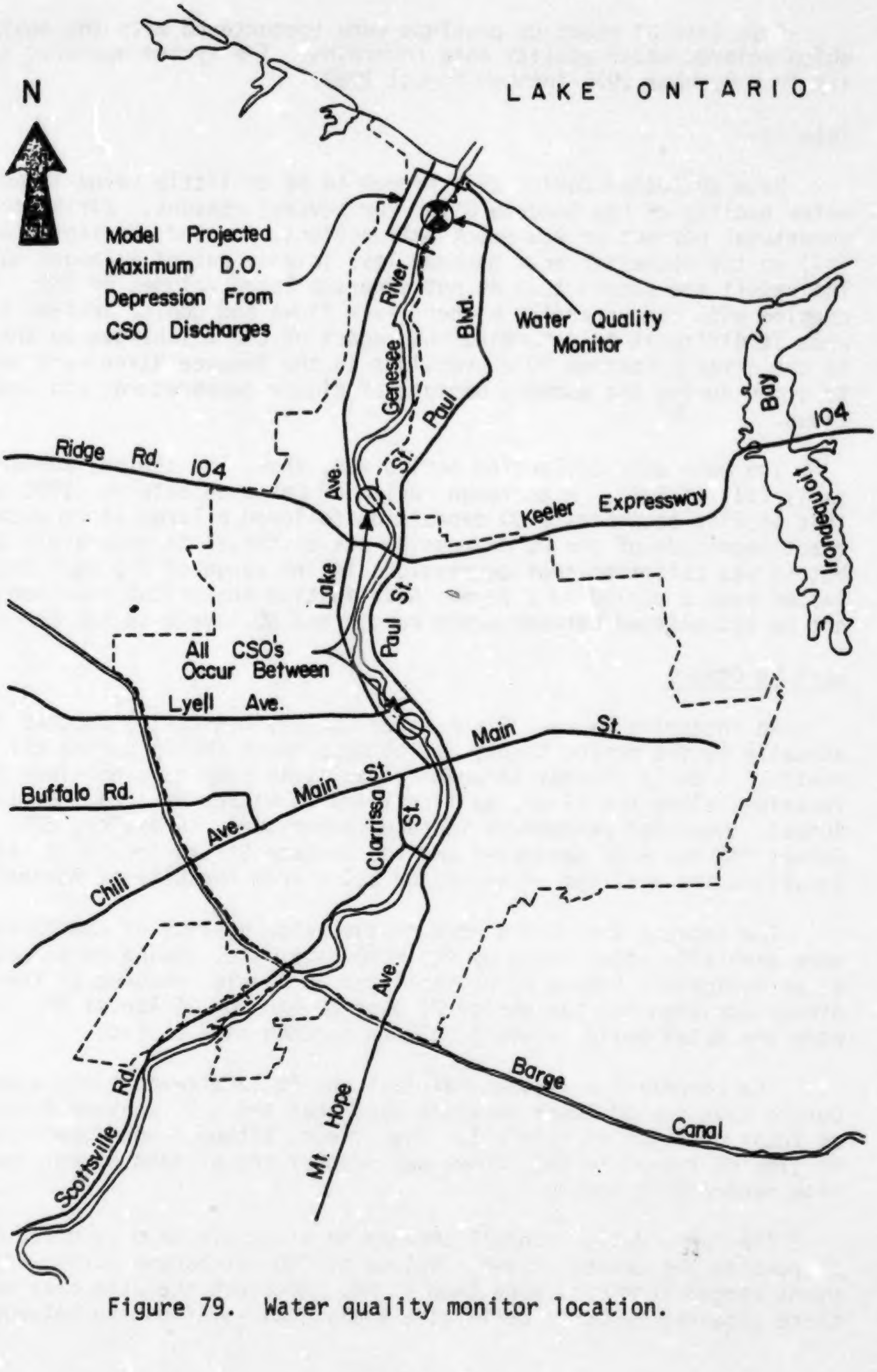

N
LAKE ONTARIO
Model Projected
Maximum D.O.
Depression From
CSO Discharges
Water Quality
Monitor
Genesee River
St. Paul Blvd.
Bay
Irondequoit
104
Ridge Rd.
104
Keeler Expressway
Lake Ave.
Paul St.
All CSO's
Occur Between
Lyell Ave.
Main St.
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
Mt. Hope Ave.
Barge Canal
Scottsville Rd.

Figure 79. Water quality monitor location.

Some initial start-up problems were encountered with the equipment, which delayed water quality data recording. The system operated satisfactorily from October 1979 through August 1980.

Results--

Data collected during 1979 proved to be of little value in assessing the water quality of the Genesee River for several reasons. First, there were occasional periods of equipment malfunctions. Second, during late fall, rainfall in the Rochester area becomes less intense but of a longer duration. The result are storms that do not generate large volumes of CSO. This fact, coupled with the generally higher river flows and cooler ambient temperatures made it difficult to determine the impact of CSO discharges on the DO levels in the river. Maximum DO depressions in the Genesee River were most likely to occur during the summer, because of higher temperatures and low river flows.

The main data collection period was, thus, the spring, summer, and early fall of 1980. A thorough review of measured data for 1980 indicated that on five occasions a DO depression followed a large storm event. The exact magnitude of the DO depression was difficult to accurately determine, but it was estimated that depressions in the range of 2.0 mg/l occurred and lasted over a period of 2 days. A definitive analytical relationship could not be established between urban runoff and DO levels in the Genesee River.

Work by Others

An intensive Genesee River water quality monitoring program is conducted annually by the Monroe County Health Department (MCHD) during the summer months. A daily (Monday through Friday) grab sample is obtained from eight locations along the River, as identified in Figure 80, from May through August. Measured parameters include temperature, turbidity, BOD, and DO. Except for the RG&E Headgates and the Stutson Street locations, all sampling locations are upstream of the major CSO's from the City of Rochester.

The impacts that CSO's have on the water quality of the Genesee River were partially established by the MCHD data base. Using fecal coliform (FC) as an indicator, Figure 81 presents the FC levels measured at the Stutson Street location for the period 25 June 80 through 26 August 80. Also plotted were the total daily rainfall volumes for the same period.

The correlation between rainfall and FC levels was quite apparent. During this period there were six days that the U.S. Weather Bureau reported at least 0.50 in. of rainfall. The effect, although not immediate because of time of travel in the river, was seen at the Stutson Street location by high recorded FC levels.

The large daily rainfall amounts on these six days resulted in CSO discharges to the Genesee River. Volume of CSO discharged during any one storm event ranged from 8 to more than 27 MG. Although the data base was limited, there appeared to be no definitive analytical relationship between total

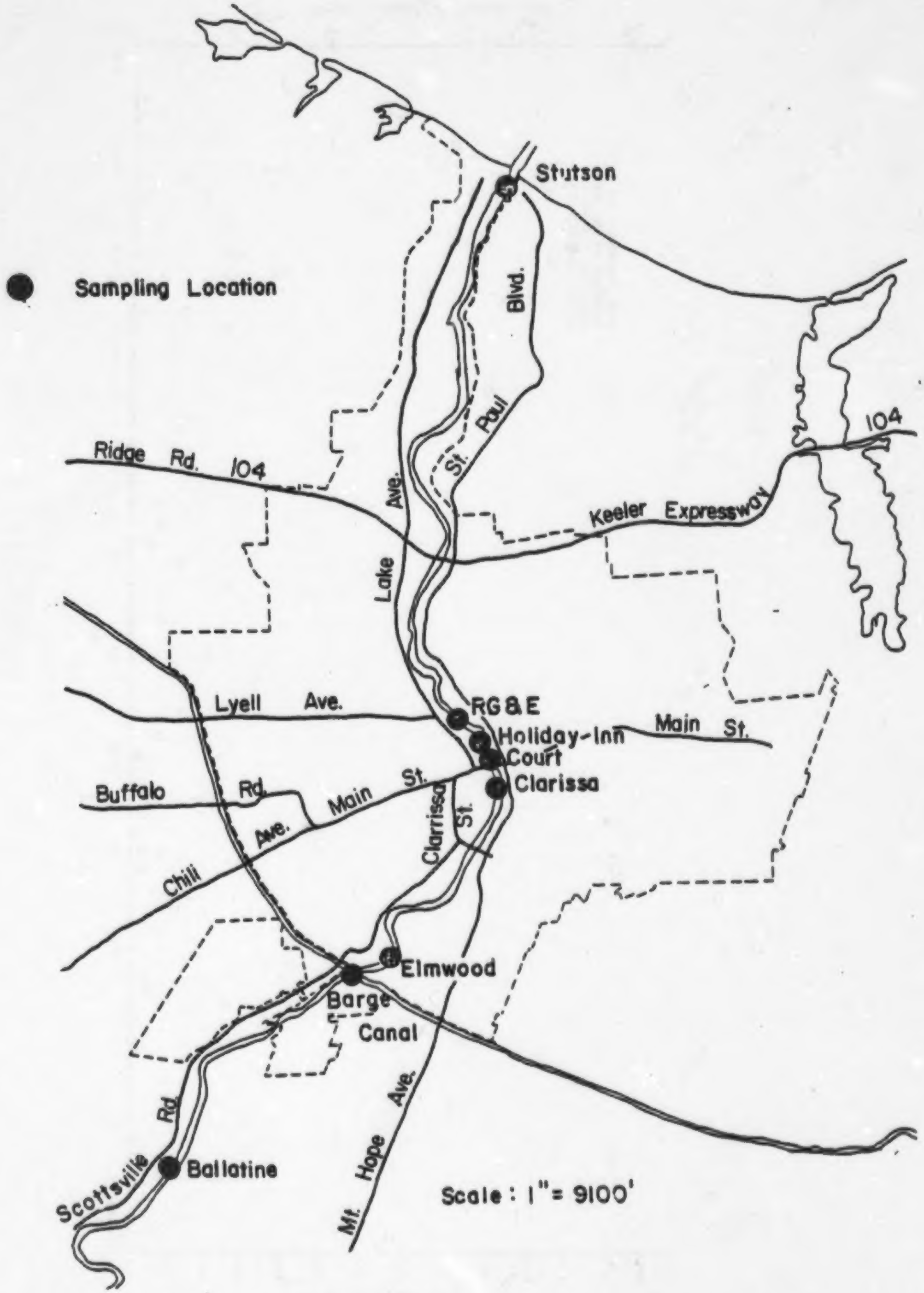

● Sampling Location
Stutson
Blvd.
St. Paul
Ridge Rd. 104
104
Lake Ave.
Keeler Expressway
Lyell Ave.
RG&E
Holiday-Inn
Main St.
Court
Clarissa
Buffalo Rd.
Main St.
Chili Ave.
Clarissa St.
Elmwood
Barge Canal
Scottsville Rd.
Ballatine
Mt. Hope Ave.
Scale: 1" = 9100'

Figure 80. Monroe County Health Department river sampling stations.

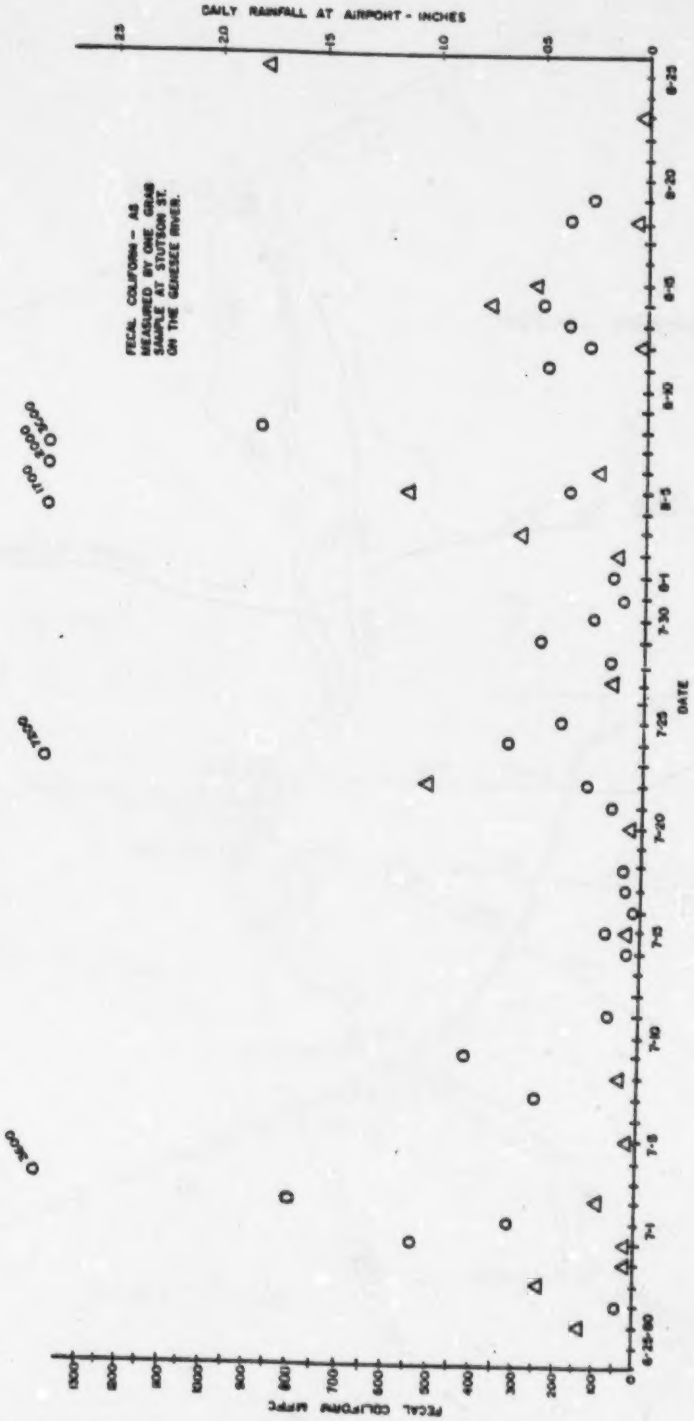


Figure 81. Fecal coliform concentrations in the Genesee River as measured by Monroe County Health Department.

overflow volume and FC levels in the river; however, a causal relationship was evident.

Indications were that the FC loads entering the river must result from CSO discharges as illustrated in Figure 81. Measurements taken at the upstream sampling stations, prior to the CSOs, indicated that high levels of FC bacteria were not present upstream of the city. DO measurements taken at the same sampling locations indicated that CSO's affect the lower reaches of the Genesee River, but a quantitative cause and effect relationship could not be established.

Fecal coliform levels in the lower reaches of the Genesee River near Lake Ontario are an important consideration in water quality improvements efforts being made by the BMP and other pollution abatement programs in the County of Monroe. The MCHD continually monitors FC levels to insure safe contact recreational use of these waters. Public bathing beaches along the shore of Lake Ontario in the vicinity of the Genesee River are an important asset to the community. The MCHD has the authority to close the beaches depending on actual FC levels measured.

Table 58 presents the number of beach closing days for a five year period.

TABLE 58. BEACH CLOSING DAYS*

Year	Days Open	Days Closed	% Open
1976	62	23	73
1977	58	18	76
1978	73	4	94
1979	72	6	92
1980	68.5	4	94

* Note: Data supplied by the MCHD

The exact relationship between CSO volume and FC levels, and therefore beach closing days, has not yet been analytically established. It has been suggested that problems of this nature occur whenever there are high flowrates in the Genesee River, regardless of the CSO's (41). More detailed studies and evaluations are required to fully assess the problem. As illustrated in Figure 81, however, a qualitative trend exists between CSO discharges and FC levels in the river.

Genesee River Water Quality Modeling

The following discussion presents the water quality modeling framework utilized to evaluate CSO impact on the Genesee River and to determine any improvement in water quality as a result of implementation of the identified BMP measures.

The modeling framework used for the simulation and analysis of Genesee River water quality was the LIMNO/SS. This mathematical model was a modification of the steady-state AUTO-QUAL Modeling System (40). It is a one-dimensional, second-order finite difference mathematical description of the river, using a series of completely mixed batch reactions to simulate the existing conditions of plug flow with dispersion. The structure of the model was based on the continuity equation and included terms for advection, dispersion, and reaction. The relationship can be described by the equation:

$$\frac{dc_i}{dt} = E\frac{d^2c_i}{dx^2} - \frac{dc_i}{dx} + R$$

where,

c = concentration of parameter i
t = time
x = distance
U = velocity
R = reaction terms and other sources or sinks of parameter i
E = dispersion.

The model was used to simulate and project conditions of carbonaceous and nitrogenous BOD, DO and FC in the Genesee River.

A prototype storm event occurring on November 10, 1975 was utilized to prepare a series of sensitivity curves relating various maximum storm induced water quality parameters in the Genesee River as a percentage of loading represented by the prototype storm event. The prototype event involved a total rainfall of 0.38 in. which occurred over a span of 8 hr with a maximum hourly intensity of 0.22 in., as recorded at the Monroe U.S. Weather Bureau. The rainfall hyetograph for this event is presented as Figure 82. The maximum rate of CSO discharge was estimated at 154.4 cfs (99.7 mgd) during the wet-weather event.

The input data set utilized to project the receiving water quality under wet-weather conditions included the principal CSO's, stormwater discharges, and steady-state discharges as presented in Table 59. These maximum discharge rates, when modeled in a steady-state framework, represented the maximum receiving water impact associated with the specific event.

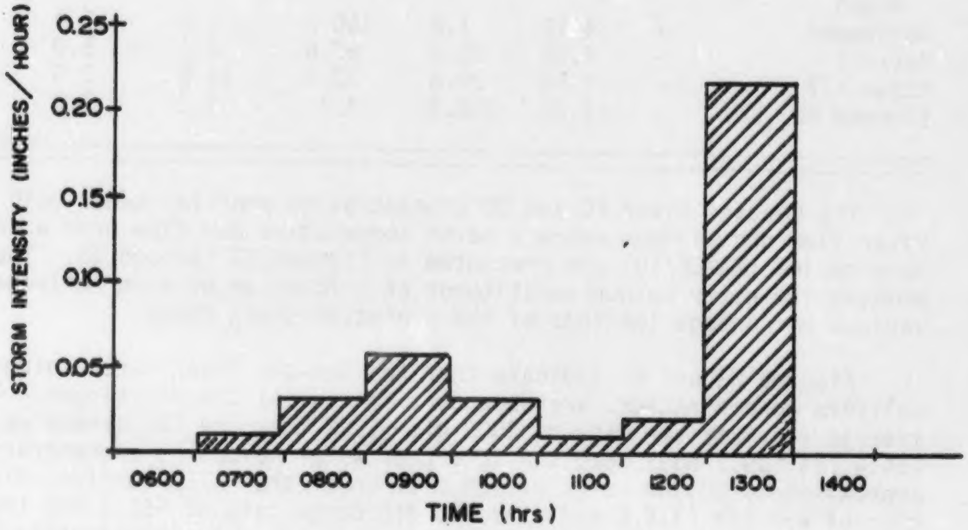


Figure 82. Rainfall hyetograph of prototype storm event.

TABLE 59. TREATMENT PLANT AND COMBINED SEWER DISCHARGES AND UPSTREAM CONDITIONS AS DEFINED FOR THE PROTOTYPE WET-WEATHER EVENT

Overflow Name	Site	River Mile	Flow (mgd)	CBOD (mg/l)	NBOD (mg/l)	DO (mg/l)	Coliform (#/100 ml)
Brooks	18	10.27	10.1	210.0	4.1	5.0	7.5×10^5
Plymouth	17	10.20	3.0	279.0	19.9	5.0	8.0×10^5
Court	26	8.10	1.4	99.0	0.5	5.0	5.0×10^5
Central	25	7.53	8.9	79.5	10.8	5.0	5.0×10^5
Mill & Factory	16	7.34	14.7	282.0	19.7	5.0	8.0×10^5
Carthage	22	6.04	30.0	585.0	16.0	5.0	11.0×10^5
Lexington	8	5.92	21.3	267.0	20.2	5.0	8.0×10^5
West Side Trunk	9	5.92	6.3	237.0	19.2	5.0	6.0×10^5
Norton & Seth Green	21	5.50	2.5	183.0	16.5	5.0	24.0×10^5
Maplewood	7	4.92	1.6	150.0	6.9	5.0	6.4×10^5
Merrill	-	4.50	21.6	57.6	2.1	5.0	0.21×10^5
Kodak STP	-	4.90	29.8	32.0	91.6	3.5	8
Elmwood Rd. Br.	-	11.20	316.5	4.7	3.2	7.8	3000

The Genesee River FC and DO concentration profiles under both average river flow and minimum average seven consecutive day flow over a ten year return period (MA7CD/10) are presented in Figures 83 through 86. The exhibits present the water column constituent as a function of river mileage for various percentage loadings of the prototype storm event.

Figures 83 and 84 indicate that the Genesee River water column fecal coliform concentrations are dependent on minimal CSO discharges. Under average yearly river flow of 2000 cfs (1292 mgd) the CSO discharge rate of 154.4 cfs (99.7 mgd) resulted in a peak water column FC concentration of approximately 20,000 colonies/100 ml. Under the MA7CD/10 river flow condition of 490 cfs (316.5 mgd) the CSO discharge rate of 154.4 cfs (99.7 mgd) resulted in a peak water column FC concentration of approximately 150,000 colonies/100 ml.

The FC concentrations predicted within the Genesee River are most important relative to the best usage of the contact and secondary contact recreational resources in the lower reaches of the river and along the Rochester Embayment of Lake Ontario. Figure 87 presents the location of presently utilized public beaches and boat dockage facilities within the impacted area.

Figures 85 and 86 present the receiving water quality model projections of the water column dissolved oxygen concentrations under wet-weather conditions under the Genesee River average yearly flow and the MA7CD/10 flow regime. The data indicated that under average river flow conditions at water column temperatures of 20°C, a CSO discharge rate of 154.4 cfs (99.7 mgd) will result in a reduction in the minimum DO concentration of approximately

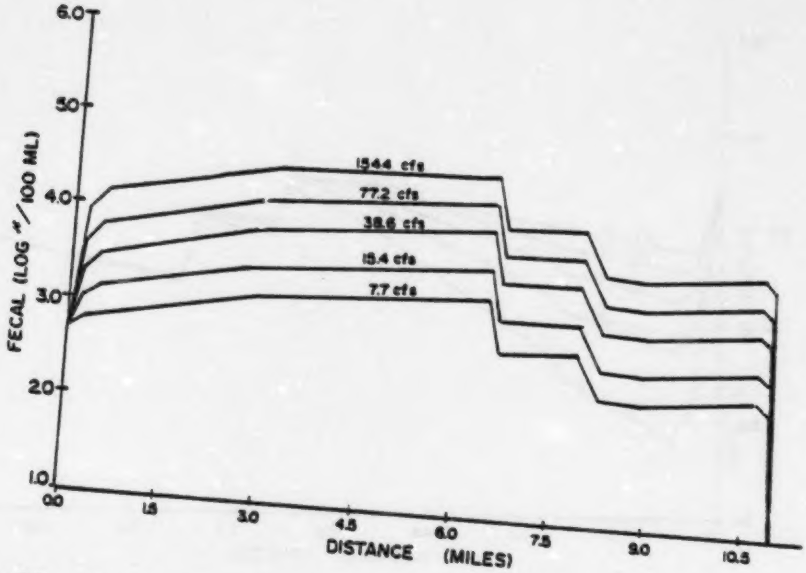


Figure 83. Model calculation of Genesee River fecal coliform in response to CSO Loads under average annual flow conditions.

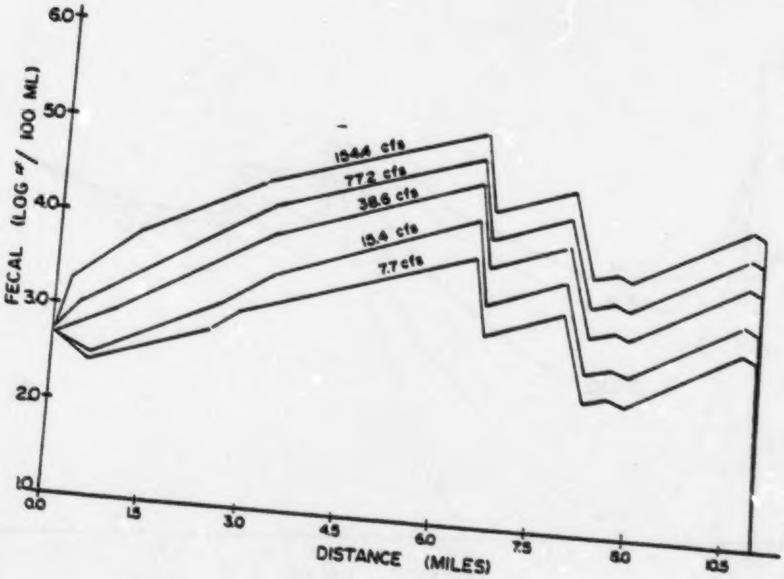


Figure 84. Model calculation of Genesee River fecal coliform in response to CSO loads under Q7-10 flow conditions.

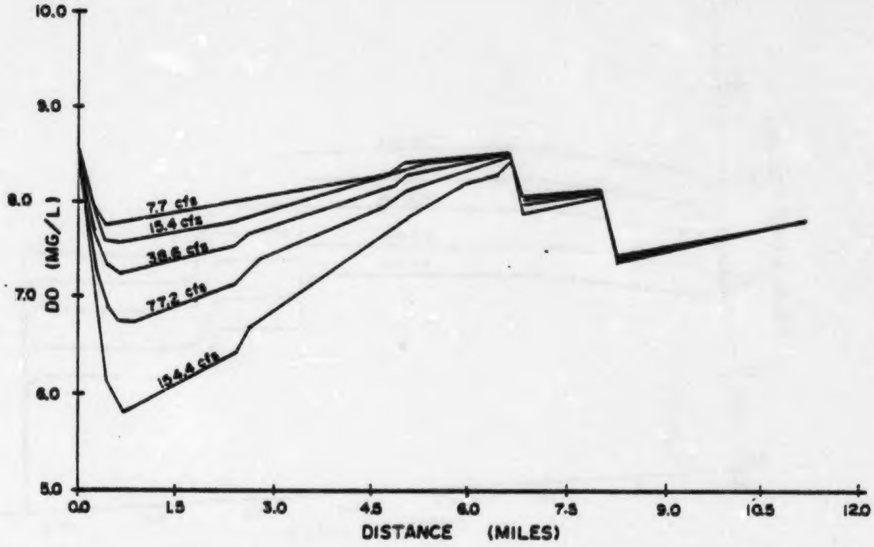


Figure 85. Model calculation of Genesee River dissolved oxygen in response to CSO loads under average yearly flow.

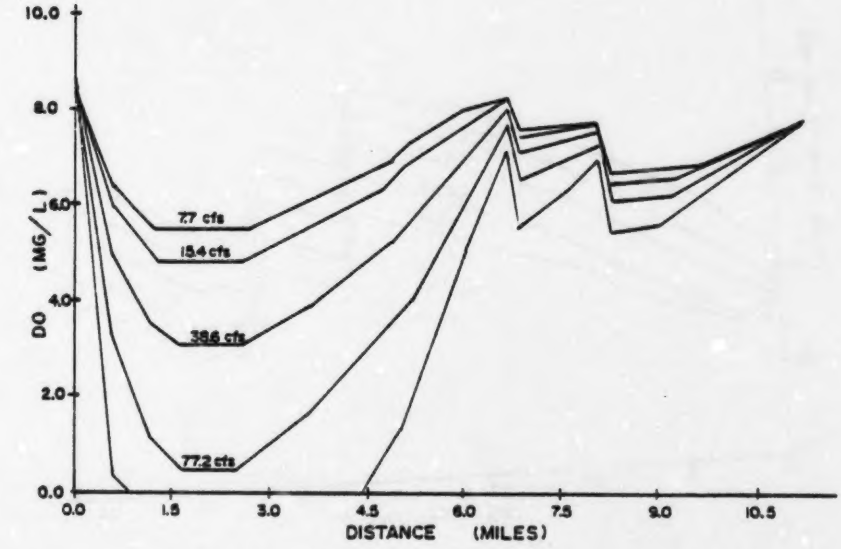


Figure 86. Model calculation of Genesee River dissolved oxygen in response to CSO loads at Q7-10.

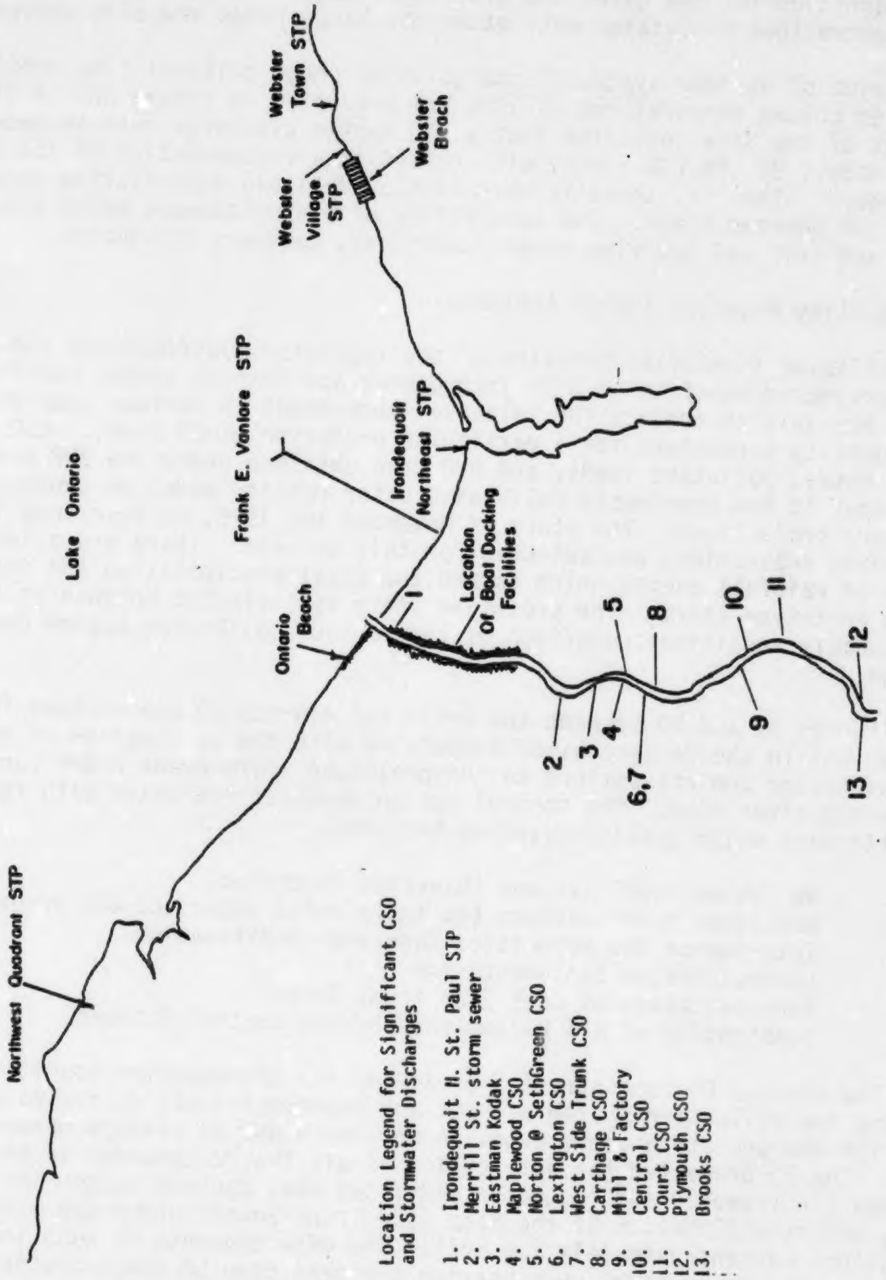

Northwest Quadrant STP
Lake Ontario
Ontario Beach
Location Of Boat Docking Facilities
Frank E. Vanlare STP
Irondequoit Northeast STP
Webster Village STP
Webster Beach
Webster Town STP
1
2
3
4
5
6,7
8
9
10
11
12
13
Location Legend for Significant CSO and Stormwater Discharges
1. Irondequoit N. St. Paul STP
2. Merrill St. storm sewer
3. Eastman Kodak
4. Maplewood CSO
5. Norton @ SethGreen CSO
6. Lexington CSO
7. West Side Trunk CSO
8. Carthage CSO
9. Mill & Factory
10. Central CSO
11. Court CSO
12. Plymouth CSO
13. Brooks CSO

Figure 87. Location of CSO discharges and recreation facilities within the impacted area of the Genesee River and the Rochester embayment of Lake Ontario.

3.0 mg/l. This represented a steady-state discharge worst case receiving water condition for the given CSO discharge rate. The projected steady-state DO concentrations associated with other discharge rates are also presented.

A plot of various system discharge rates under critical flow conditions and water column temperatures of 20°C are presented in Figure 88. A close analysis of the data indicated that a CSO system discharge rate in excess of approximately 25 cfs (16.2 mgd) will result in a contravention of the 4.0 mg/l DO standard. That is, there is very little available assimilative capacity within the Genesee River. The possibility of a simultaneous major CSO discharge and critical low flow river conditions, however, is remote.

Water Quality Modeling Impact Analysis--

Utilizing identified benefits of the regulator modifications and the model projected benefits of both interceptor and control system modifications, it was possible to project the relative improvement in various receiving water quality parameters for a particular prototype storm event. CSO discharge rates, pollutant loads, and SOD data obtained under the BMP program were input to the previously calibrated water quality model to generate the necessary projections. The storm of November 10, 1975, as described in the preceeding subsection, was selected for this purpose. There are a large number of rainfall events which exceed the total precipitation and intensity of the prototype storm. The prototype storm was selected because it allowed a reasonably sensitive comparison of the various collection system control options.

Figures 89 and 90 present the projected minimum DO and maximum FC concentrations in the Genesee River associated with the application of the various system control options to the prototype storm event under conditions of average river flow. The control option measures evaluated with regard to the attendant water quality response included:

- No System Modifications (baseline condition)
- Regulator Modifications (as implemented under the BMP program)
- Interceptor and Associated Regulator Modifications
- Control System Implementation
- Rehabilitation of East Side Trunk Sewer
- Combination of All Recommended System Control Options

The minimum DO concentration projected for the baseline condition representing the collection system without any improvement was approximately 5.8 mg/l for the prototype storm event under conditions of average Genesee River flow. The DO projected for application of all the recommended system control options (interceptor and regulator modifications, control system implementation, and rehabilitation of the East Side Trunk Sewer) under the same set of conditions was approximately 7.8 mg/l. The effectiveness of each individual component can be assessed by comparing the predicted DO concentration against the baseline condition.

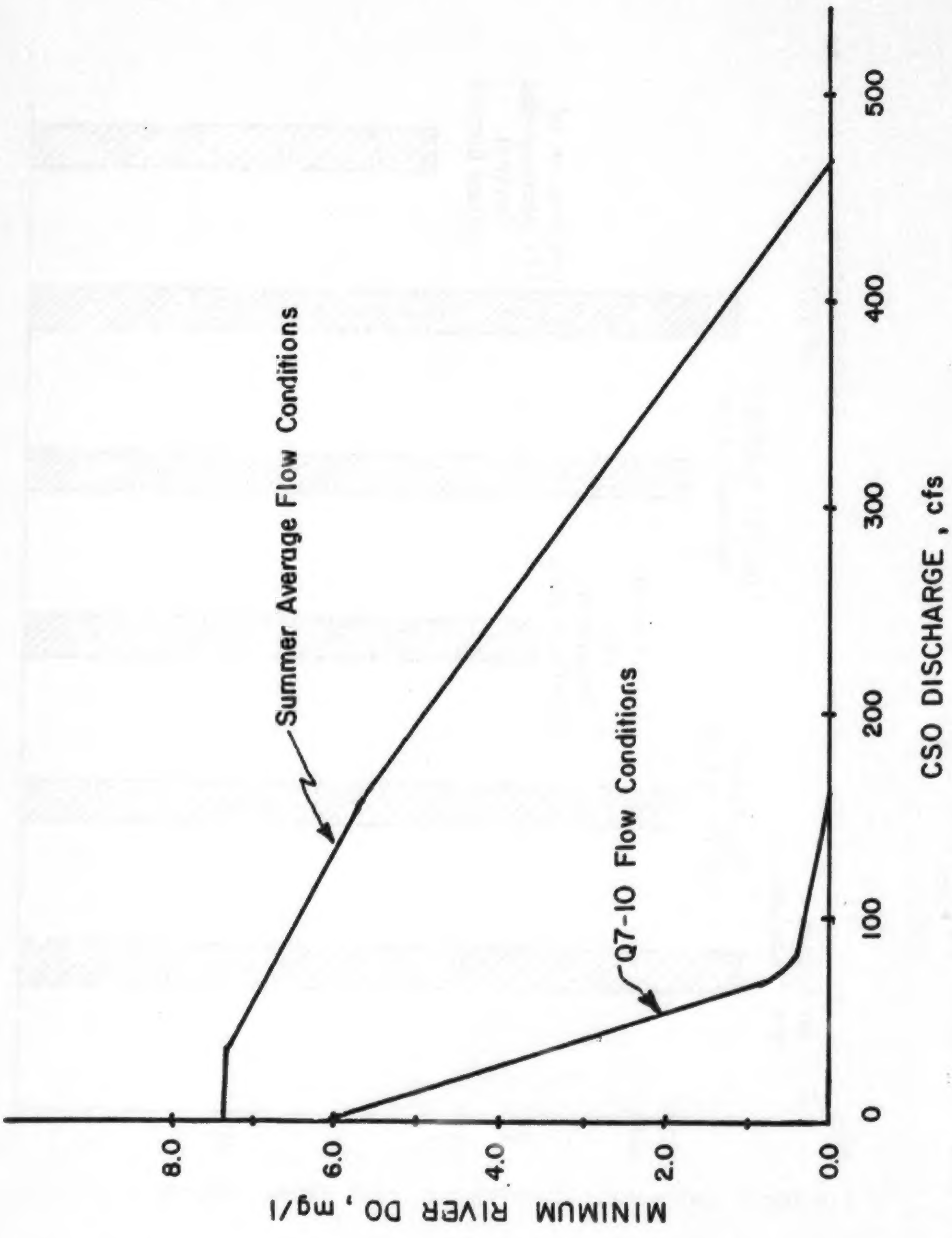


Figure 88. Critical dissolved oxygen concentrations vs. magnitude of CSO loading.

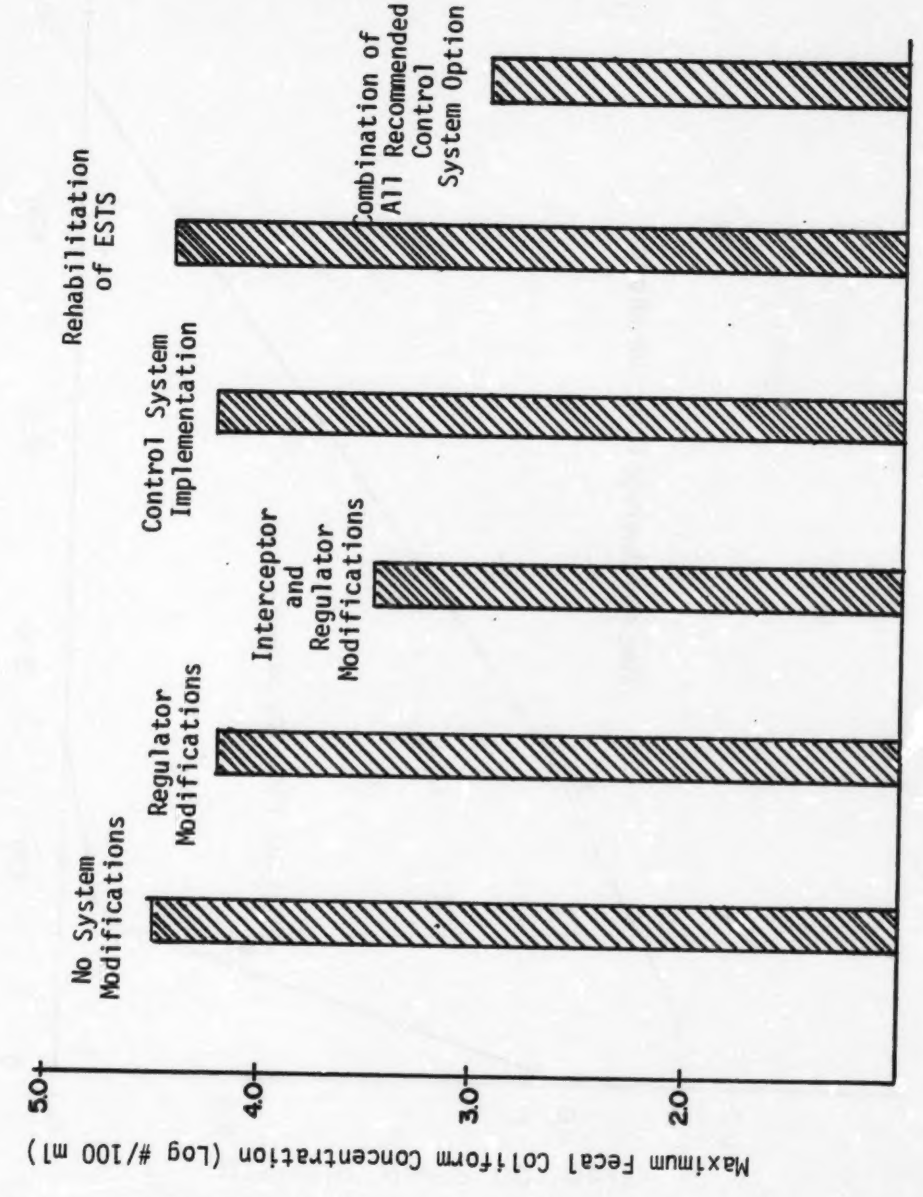


Figure 89. Projected maximum fecal coliform concentration in the Genesee River associated with the application of various control system options to the prototype storm event under conditions of average river flow.

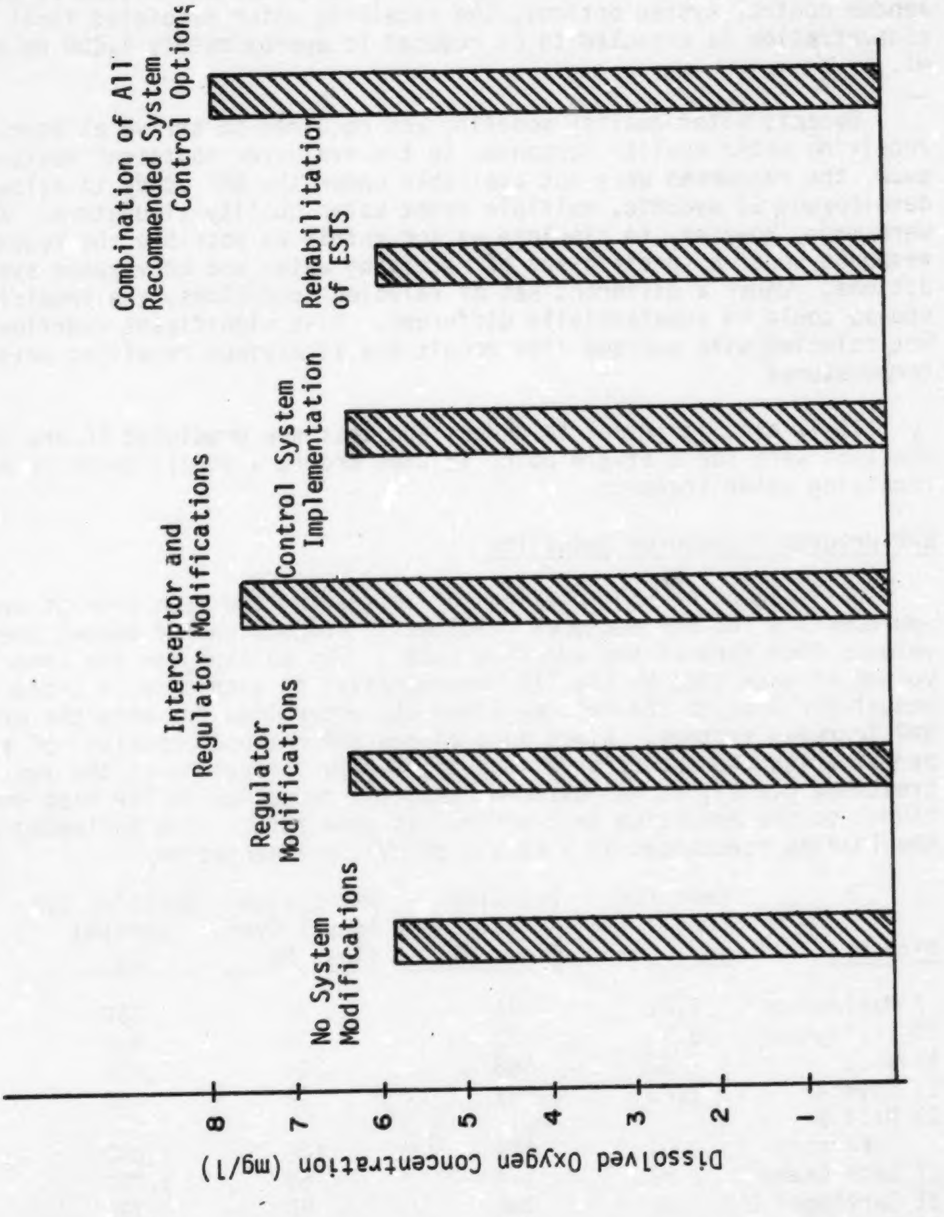


Figure 90. Projected minimum dissolved oxygen concentration in the Genesee River associated with the application of various system control options to the prototype storm event under conditions of average river flow.

The baseline condition relative to the predicted fecal coliform concentration in the Genesee River under the influence of the prototype storm event is approximately 30,000 colonies/100 ml. Upon implementation of all recommended control system options, the receiving water predicted fecal coliform concentration is expected to be reduced to approximately 1,000 colonies/100 ml.

Dynamic water quality modeling was required to arrive at appropriate receiving water quality responses to the evaluated abatement measures however, the resources were not available under the BMP study to allow for development of dynamic, multiple event water quality simulators. Attempts were made, however, to simulate as accurately as possible the receiving water response under a specific set of receiving water and conveyance system conditions. Under a different set of selected conditions, the predicted response could be substantially different. Most significant overflow events do not coincide with average flow conditions at maximum receiving water column temperatures.

It is also important to understand that the predicted FC and DO concentrations were for a single point in time and at a single point in a specific receiving water transect.

BMP Program Phosphorus Reduction

Utilization of SSM modeling for assessing the reduction of overflow by implementing the BMP measures resulted in projections of annual overflow volumes from each of the overflow sites. By multiplying the annual overflow volume at each site by the TIP concentration at each site, a projection of annual TIP load to the Genesee River was determined for both the existing and BMP improved systems. Since none of the BMP measures consist of treatment per se (with the exception of the split-flow operations at the Van Lare treatment plant), it was assumed that the reduction in TIP load was proportional to the reduction in overflow at each site. The following presents the loading reductions as a result of BMP implementation.

Site No.	Mean TIP Conc. mg/l	Existing Annual Overflow, MG	BMP-System Annual Overflow, MG	Existing TIP Loading lbs	BMP TIP Loading lbs
7 Maplewood	1.21	84	79	850	800
10 Lexington	0.95	73	39	580	310
11 WSTS	2.33	588	309	11,430	6,000
17 Spencer	1.28	62	32	660	340
21 Mill & Factory	1.01	129	106	1,090	890
27 Seth Green	1.22	174	69	1,770	700
31 Carthage	2.32	144	90	2,790	1,740
36 Central	0.78	186	80	1,210	520
Total		1,670	990	23,120	13,520

As indicated in the above tabulation, an estimated 41.5 percent reduction in TIP loading to the Genesee River from CSO is projected to occur upon implementation of the BMP measures. However, it must also be pointed out that the additional flow contained within the collection system as a result of BMP improvements is transmitted to the Frank E. Van Lare treatment plant for processing. A certain fraction of the additional TIP transmitted to Van Lare will be discharged in the plant effluent. It is therefore necessary to examine the performance of the Van Lare plant under the split-flow mode of operation in order to assess the actual realized reduction of TIP loadings as a result of BMP improvements.

For the period of split-flow operation utilizing partial chemical-assisted primary settling treatment (February 18, 1979 through June 3, 1979) TIP removals averaged approximately 65 percent, with an average plant effluent concentration of 0.86 mg/l. Based on a retention in the conveyance system of an additional 9600 lbs/yr of TIP as a result of BMP implementation, and an average removal efficiency of 65 percent at the Van Lare plant, an overall reduction in wet weather phosphorus loading from CSO's to the Genesee River and Embayment area of 6240 lbs/yr, or 27 percent, can be realized.

SECTION 9

BMP PROGRAM IMPLEMENTABILITY

COMBINATIONS OF BMP OPTIONS

Evaluation of various control measures investigated under the overall BMP program indicated that a combination of management options - both source control and collection system - were most effective in maximizing the use of the existing system and minimizing the frequency and volume of CSO's. Source control management options which appeared to be the most effective measures for pollution abatement were surface flow attenuation, stormwater inlet control, and porous pavement.

Effective collection system management options included a combination of improved overflow regulators/weirs and minimal structural improvements to the St. Paul Boulevard Interceptor (SPBI) to provide for optimized system control and minimized CSO's.

ANTICIPATED CSO REDUCTIONS

Based on overflow monitoring conducted prior to and after implementation of the identified regulator/weir modifications, a measureable reduction in the frequency and volume of CSO discharged to the Genesee River was realized. System modeling further indicated that implementation of the identified SPBI improvements, selective trunk sewer rehabilitation, and installation of collection system control structures would reduce the average annual CSO volume by 41%. For these same improvements, average annual CSO duration would be reduced by 46%.

The effectiveness of the implemented and proposed BMP control measures on CSO reduction for an individual storm event is directly related to the magnitude and intensity of rainfall. The greater the rainfall intensity, the less effective are the BMP control measure in reducing CSO's. Monitored data and system modeling indicated that for storm events involving rainfall volumes of greater than 0.5 in., the Rochester BMP measures would reduce CSO volume by approximately 3%. Storms of this magnitude occur about 20 times per year in the Rochester area. Storms containing 0.5 in. or less of rainfall occur, on the average, 66 times per year.

In any event, implementation of minimal structural BMP collection system management options will result in CSO reductions only if the existing collection and treatment system components are presently under-utilized. If these systems are operating at their maximum capacities, then the identified BMP solutions will not be applicable.

Relative to source control measures such as porous pavement, inlet control, street sweeping, and sewer and catchbasin cleaning, the average annual CSO percent reduction upon implementation of or increased such activities cannot be readily determined. It is known, however, that implementation of these source control measures will reduce the annual volume of CSO but the magnitude of the reduction cannot be determined. Source control measures such as selective use of porous pavement and installation of inlet control devices can result in significant CSO reductions in certain problem areas. Effectiveness can only be determined on a site-specific basis.

Although increasing the frequency of street sweeping operations can possibly result in additional pollutant removal, increased costs are almost proportional to the percent increase in sweeping frequency. That is, since street sweeping operations are labor intensive, increasing the street sweeping frequency by a factor of two, for example, will result in an incremental cost increase of approximately 100%. An analytical relationship between pound of pollutant removed and number of passes and frequency of sweeping has not been established. From the Rochester BMP study, however, the marginal increase in pollutant removed was small for doubling and tripling the frequency of street sweeping operations. Therefore, because of the large marginal cost to marginal benefit ratio, increased street sweeping activities was not considered a viable, effective pollution control method. A similar relationship exists for increased catchbasin cleaning activities.

The use of porous pavement is one source control management option that appears to be a cost-effective solution to reducing the rate of stormwater in selected areas. If the soil permeability conditions are adequate, thereby eliminating the need for an underdrain system, costs for porous pavement are comparable to those for conventional asphaltic pavements. The Rochester BMP study evaluated the use of porous pavement under parking lot applications. Results indicated that such pavements can adequately support the imposed structural loads while maintaining a high rate of water infiltration. Extrapolation of the use of porous pavements to other areas such as roadways should not be made. Site-specific investigations should be conducted to evaluate the use of porous pavements in applications other than for parking lots.

If increased surface ponding can be tolerated, then implementation of inlet control concepts can be an effective method of reducing CSO's. By keeping stormwater out of the collection system or by limiting its inflow to an acceptable rate, increased downstream flowrates can be minimized. The costs involved in installing inlet control devices, such as Hydro-Brakes, are small, however, the effectiveness of these devices has been shown only for small problem areas. The feasibility of their use for large urban areas must be more fully evaluated on a site-specific basis. In any event, additional surface ponding will result from the installation of inlet control devices, and therefore, the impacts of increased flooding must also be fully evaluated.

COSTS AND FINANCING

The Rochester BMP program involved the implementation of various minimal structural control measures along with the installation of necessary overflow and water quality monitoring to evaluate their effectiveness. Tables 60 and 61 present a summary of the costs associated with various aspects of the overall study.

TABLE 60. PROGRAM ELEMENT COSTS

Element	Cost*
Upgrading existing CSO Sites	$5000/site
Water quality monitoring system	$15,000
Upgrading of existing PDP-8E computer system	$18,000
Regulator/Weir modifications	$1,500/site
Hydro-Brake Regulator	$10,000
In-system monitoring site	$7,000/site
Maintaining all monitoring systems	$700/wk

* Note: These costs include acquisition of the necessary equipment and manpower to implement all instrumentation. Costs for computer upgrading includes development of necessary software.

TABLE 61. BMP SYSTEM IMPROVEMENT COSTS

System Improvement	Cost (mil $)
SPBI Improvements	11.0
ESTS Rehabilitation	6.0
Control Structures	3.5
Total	20.5

Note: These costs are total project costs in mid-1982 dollars and include items such as engineering, legal and miscellaneous (29).

The method of financing such projects is expected to be consistent with past practices by the County of Monroe. The legal and administrative procedures required to obtain the necessary funding (local share) for the program are summarized as follows (2):

1. The Facility Plan is submitted to the Rochester Pure Waters Administrative Board for their approval and adoption as a basis for future District improvements.

2. When the decision is made to proceed with the projects, and when eligibility for state and federal aid is obtained, plans and specifications are prepared for each project based on the Facility Plan data for submission to the County Legislature and the New York State Department of Audit and Control to obtain authority for the sale of bonds.

3. The County Legislature refers the plans and specifications and recommendations to the Public Works and Ways and Means Committees for review and recommendation to the full Legislature.

4. Upon receipt of the recommendations of the Public Works and Ways and Means Committees, the Legislature adopts a resolution calling for a public hearing on the proposed project based on a notice published in the official newspapers of the county not less than ten nor more than twenty days before the day set therein for the hearing.

5. After consideration of the results of the public hearing the Legislature adopts a preliminary resolution approving the proposed increase and improvement of facilities subject to receiving the consent of the New York State Department of Audit and Control.

6. The County Manager submits an application to the Department of Audit and Control for the Comptroller's order of consent.

7. Upon receipt of the Comptroller's order of consent the Legislature, on recommendation of the Ways and Means Committee, adopts a bond resolution, which resolution is published in the official newspapers of the County with twenty-day legal notice of estoppel. Upon expiration of the estoppel period the District Administrative Board is in position to authorize implementation of the program including additional engineering as required.

SCHEDULE OF IMPLEMENTATION

The intent of the Rochester Best Management Practices Implementation Program was not to show whether CSO's are responsible for impaired receiving water quality degradation but rather to investigate the possibility of implementating various source and control management options to alleviate known problems caused by periodic CSO discharges. As a result of the study, several minimal system management options were identified as cost-effective in reducing the frequency and volume of CSO as determined on an annual basis. These include the SPBI improvements, ESTS rehabilitation, and control structures. The identified measures are compatible with other ongoing abatement programs and form an integral part of the overall Master Plan for Monroe County (2).

By the nature of a BMP oriented program, pollution abatement can serve as a Phase 1 solution to the identified CSO problem that can be initiated almost immediately while long-term design and construction of more structurally intensive abatement alternatives are undertaken.

Those BMP measures demonstrated to be cost-effective also involve items with varying schedules for implementation. By carefully configuring the implementation of the most promising BMP alternatives, both short and long range objectives can be met. Most importantly, however, is the establishment of these objectives.

The short range objective is simply to minimize the frequency and volume of CSO presently discharged to the Genesee River through the full implementation of minimal regulator and weir modifications as described herein. The effect will be a reasonable reduction in CSO for the more frequent, less intense storm events. Thus, these modifications will be most effective during the spring and fall months because of the general rainfall patterns in Rochester characteristic of these months.

A substantially larger reduction in CSO will be realized upon the implementation of interceptor improvements and the installation of various control structures at selected locations. The effect will be seen over a wider range of storm events than those associated with the minimal regulator/weir modifications. This forms the basis for the long-term objectives.

It should be realized that even after these minimal structural improvements, CSO discharges to the Genesee River will still be substantial for the more intense, less frequent storm events, such as those associated with the summer months. The other ongoing pollution abatement programs address basically these types of storm events (2). Implementation of the identified effective BMP measures is critical to the timely reduction of CSO discharges to the Genesee River. Work associated with these projects can proceed concurrently with all other ongoing programs and, in fact, will complement the other ongoing abatement programs. Figure 91 presents the schedule of implementation for the BMP measures and their relationship to other abatement programs.

A brief clarification of the differences between BMP, minimal structural, and structurally intensive abatement alternatives should be made. There are no rigid criteria that determine into which of these categories a particular management option falls. There are guidelines, however, which are easily applied. In general, any source control measure identified previously in Figure 2 is classified as a BMP option. Most collection system management options, as previously shown, are generally termed minimal structural. That is, they do not involve a large capital expenditure. Structural intensive alternatives are generally those traditional measures that involve the application of storage and treatment systems. For example, tunnels and overflow treatment facilities would be considered structurally intensive. In general these structural programs are more expensive than BMP programs, however, pollution abatement resulting from these capital-intensive programs is much greater.

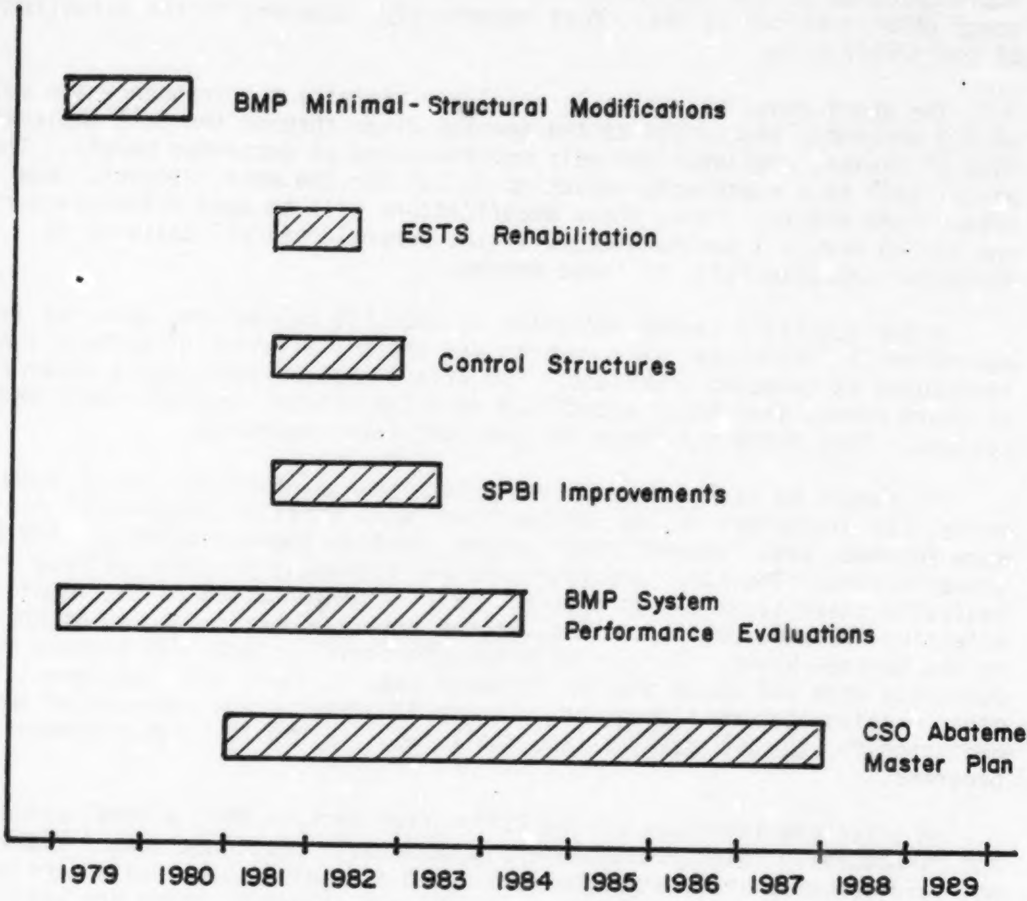


Figure 91. Relationship of BMP improvements to overall CSO Abatement Master Plan for the Rochester Pure Waters District.

It is obvious that what is expensive to one municipality may not be that expensive to another. Costs are totally relative and must be judged from a particular point of view. Aside from legal and institutional constraints that may prevent the full implementation of various BMP or more expensive abatement measures, BMP alternatives are generally effective in pollution abatement resulting from typical or average storm events for small capital expenditures, whereas, structural alternatives are necessary to protect the receiving waters from CSO's resulting from less frequent, more intense storm events.

It is important to remember that the overall effectiveness of a BMP oriented abatement program is largely dependent on the characteristics of the existing conveyance and treatment systems involved. If the existing sewer system is already fully utilized in all aspects, those BMP measures identified as minimal structural under the general heading of collection system management become ineffective and, therefore, not recommended. In these instances, source control management options may be more effective. The effectiveness of source control management options will be realized over a narrower range of storm events. That is CSO reductions will occur for only those frequent, small intensity storms.

In terms of planning and engineering financial assistance minimal structural improvement projects for which USEPA Construction Grants monies were applied for and received were noted. For all the BMP CSO abatement measures discussed in this report, it is recommended that planning and engineering assistance in the form of USEPA Step I, II, and III grants be applied for to minimize the financial burden of such programs on the particular municipality involved. The amounts of reimbursement available is 75% federal share and 12.5% state share of the eligible monies expended.

LEGAL AND INSTITUTIONAL CONSTRAINTS

During the course of conducting the BMP investigations it became apparent that there were several source control management options that would be difficult, if not impossible, to implement. Improved and/or increased street sweeping may be difficult to implement as a source control management option because this operation is a function of the City of Rochester and not of the Monroe County Division of Pure Waters. Often, recommendations made by one governmental agency that affect a different governmental agency and political authority are simply not readily implementable. Increased street sweeping as a source control option would be difficult to implement because of this effect.

The identified minimal collection system management options identifed previously appear to be relatively easily implementable with no discernible legal and institutional constraints associated with such improvements. In general, source control management options associated with a BMP oriented program are more difficult to implement than collection system management options. The degree of difficulty in the implementation of various control measures depends on the municipality involved. The municipality must determine the relative acceptability and implementability of any BMP abatement alternative.

RELATIONSHIP TO OTHER ONGOING POLLUTION ABATEMENT PROGRAMS

As mentioned previously, the source and collection system management options demonstrated to be cost-effective are completely compatible with other ongoing pollution abatement programs. Most notable of the other concurrent programs is the West Side Tunnel Storage/Conveyance System (2). The minimal structural improvements recommended in this report address only a small number of the problems to be solved by the proposed tunnel program, especially when evaluated on a storm event basis.

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